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ASPHALT STABILIZATION OF A MARGINAL AGGREGATE

by

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ABSTRACT

This thesis was an attempt to improve the physical properties of bituminous mixtures of an aggregate normally considered unsuitable for base course construction. Two aggregate gradations were used; one partially outside the requirements for granular base course material in the region of the No.100 sieve and one completely within the specifications. Three, five and seven percent of asphalt cement and medium curing cutback asphalt were introduced in the conventional manner and in the form of a foamed asphalt.

The test used was the immersion compression test. Samples were tested in unconfined compression, dry and after soaking for four days. The amount of swell and water absorption of the immersed specimens was also measured.

In general the highest strengths were produced by the hot mixtures of the gradation with the high fines content mixed with asphalt cement. Of the two hot mixes, slightly higher strengths were produced using the foamed asphalt. Foamed asphalt cement mixed with the aggregate at optimum moisture content produced strengths lower than the hot mixes but higher than the cold mix cemented by the cutback asphalt.

From this limited investigation it appears that the foamed asphalt achieves a more uniform distribution of the asphalt cement, hence higher strengths. Gradation A mixtures appear to be the most suitable for use in a stabilized base course with higher dry and soaked strengths; however, due to its high swelling characteristics,

a pavement built upon it may suffer damage from the swelling.

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CHAPTER I

INTRODUCTION

General

The accelerating demand for roadbuilding materials and the diminishing supply of aggregates of the desired quality makes a better understanding of the performance of aggregates of various characteristics in bituminous pavements most urgent. Of great importance is the development of means of improving poor quality aggregates through various processes of beneficiation.

Recognizing this requirement, the Alberta Joint Highway Research Programme has sponsored a series of investigations into the chemical stabilization of various soils. This thesis, as part of the continuing programme will attempt to compare different methods of stabilizing a granular material with bitumen.

Bituminous Aggregate and Soil Stabilization

When employed as a stabilizing agent, bitumen is used to produce various effects on different soil types. They may briefly be described as:

a. To provide strength to cohesionless materials, such as clean sands, by acting as a binding or cementing agent;

b. To stabilize the moisture content of cohesive fine-grained soils, such as clay; and

c. To provide cohesive strength and to waterproof materials, such as gravels, which inherently possess frictional strength.

Scope

The aggregate selected for testing was considered to be of marginal quality for use in base course construction because of the high proportion of fine material present. The purpose of this thesis is to attempt to improve the physical properties of bituminous mixes using this aggregate. In this study the effect of varying the aggregate gradation, the binder type and content and the method of introduction was investigated. The gradation was altered by reducing the fines content so that it fell within normally accepted gradation requirements. A penetration grade asphalt cement and a medium curing cutback asphalt were introduced in the liquid form to produce hot and cold mixes respectively. In addition, a new process was used in which the asphalt cement was introduced as foam.

The unconfined compression test before and after immersion in water, as outlined in ASTM[a] (1) Designation D1074-60 was chosen as the method of test. A strength relationship as well as a measure of the loss of cohesion resulting from the action of the water was obtained.

Organization

Chapter II is devoted to a brief literature review. The fundamentals of asphalt stabilization, stripping tests, the immersion-compression test and foamed asphalt are considered in this review.

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[a] Numbers in brackets refer to references in the bibliography

A summary of the properties of the aggregate and asphalts used is set forth in Chapter III.

Chapter IV contains a description and discussion of the test procedure and equipment used.

The test results and a discussion of them is contained in Chapter V.

Chapter VI contains the conclusions and recommendations for future research.

CHAPTER II

LITERATURE REVIEW

Fundamentals of Asphalt Stabilization

The purpose of a bituminous stabilizing material is to provide cohesive strength by binding the particles together as well as to waterproof the mixture. The waterproofing prevents water from reaching the particle surface and causing stripping, that is loss of adhesion between aggregate and binder, as well as cutting off any capillary migration of water.

Two theories regarding the action of asphalt in waterproofing soils have been put forward by Endersby (2). First is the plug theory which contends that capillaries are plugged with globules of asphalt preventing water from entering or leaving. The other is the intimate mix theory under which the individual particles are believed to be entirely coated with asphalt. Both theories exhibit certain features which appear to prove them, but it is more likely that a combination of the two theories is actually true.

Diverse opinions exist as to the action of the binder as the cementing agent of the particles as well. One maintains that the binder exists as a continuous phase of a plastic matrix in which the aggregate particles are suspended during manufacture and placement and in which they are imbedded after compaction. The other contends that films of the binder form around the large particles, that the mineral dust is taken into suspension by the binder, and that the films coalesce and cement the aggregate together when

compacted. (3)

Of great importance when asphalt is used as a stabilizing agent is the development of a strong bond between the asphalt and aggregate. In order to develop this bond, the asphalt and aggregate must be brought into the most intimate contact possible. The ability of a liquid to make intimate contact with a solid is known as its wetting power and of a solid to make contact with a liquid its wettability. (4)

The wetting power of asphalt is largely controlled by its viscosity and surface tension. It may be increased by decreasing the viscosity by heating for example, or by decreasing the surface tension by the use of additives.

The wettability of an aggregate is controlled by a number of factors, among them the surface texture and surface tension of the aggregate. Increasing the wettability of an aggregate can be accomplished by altering the surface chemistry with additives, but this is thought by some to be impracticable. In practice it has been found that the wettability can be increased by removing impurities such as dust from the aggregate surface.

If the surface of the aggregate is covered with a film of water, adhesion of the asphalt to the aggregate is prevented unless the asphalt is able to displace the water. The lower surface tension of the asphalt may allow coverage of the water wetted aggregate surface; because of this, moisture is often used to promote coverage especially in finegrained aggregates.

The maintenance of adhesion in the presence of water is a

separate problem. Some factors which promote wetting, such as asphalt of low viscosity or aggregates with smooth surfaces, are not advantageous from the standpoint of adhesion. Aggregates which are acidic in nature, such as quartz and granites, show relatively poorer adhesion in the presence of water than basic materials such as limestone and dolomite. Aggregates with weathered surfaces may show quite different adhesion characteristics from the freshly crushed material. (5)

The amount of moisture required to promote good coverage is said to be slightly in excess of that required for optimum compaction. Increasing the moisture content improves the asphalt distribution but compaction and strength fall off and curing becomes a problem. The determination of the proper moisture content is quite cumbersome to determine experimentally and visual methods have been developed to facilitate its determination. The "exudation point" based on the combined asphalt and water content which the soil will tolerate when compacted has been used. Another method uses the "fluff point" based on the fact that as water is added, the soil reaches and passes through a condition in which it becomes loose and "fluffy" with a moist appearance.

It has been found that the degree and method of mixing have a definite effect on the properties of stabilized soils. As mixing progresses an optimum phase of waterproofing is passed through. This is probably due to breaking down of aggregates of soil particles as mixing progresses. Hence the asphalt becomes too thinly spread

over the particle surfaces for good waterproofing.

Winterkorn (6) has extended and modified the principles of granulometry as developed in connection with concrete work to apply to bituminous soil stabilization. If the binder is essentially a high viscosity liquid such as most bitumens, the strength of granular mixes is said to be dependent on the absolute volume occupied by the granules which form the bearing skeleton. The higher the volume occupied by the aggregate, the greater is the strength. This, not gradation, is said to be the most important factor in the design of bituminous mixes with granular bearing skeletons.

It was reported that beach sands stabilized with strong resinous cementing agents followed the law

$$S = K \left[\frac{C}{1-s}\right]^{n}$$

in which

S = compressive strength,

K = a constant, essentially the strength of the cement, but influenced by materials and packing,

C = absolute volume of the sand,

s = absolute volume of the cement, and

n = constant depending on the material and geometric factors.

Values of these constants were listed for various types of sands and a design procedure outlined based on a logarithmic plot of the cement void ratio versus compressive strength. This plot

exhibited a straight line relationship for the various sand types used. This enabled binder contents to be selected which satisfied the cement/void ratio required for the design strength plus that required for surface absorption of the sand.

The Development and Use of Foamed Asphalt

Development. In order to accomplish an intimate mixture of asphalt and aggregate it is necessary to render the asphalt temporarily fluid. Conventional means by which asphalt cement may be brought to workable fluidity are by heating, by diluting with a petroleum solvent or by emulsifying the asphalt in water. Another method of achieving fluidity has recently been developed utilizing the properties of foamed asphalt. (7)

It has been reported by the developers of this process that when asphalt foams it expands, creeps and flows, remains soft at low temperatures and has very high adhesive properties. Because of this expansion, better distribution is achieved than with a similar amount of conventially mixed asphalt. Due to low viscosity of foamed asphalt, mixing may be performed at lower temperatures. Since the binder remains soft for some time as a foam, the mix can be laid at lower temperatures. The high cohesive and adhesive properties of the foam improves adhesion between binder and aggregate. It is said that the modified surfaces tension of the foam enables it to displace surface moisture on the aggregate.

When the bubbles of the foam break, thin films of asphalt cement are available to coat particles of aggregate on contact. The high penetrating power of the foam allows it to penetrate lumps of dust which liquid asphalts would coat.

The production of the foam is achieved by injecting saturated steam into the heated asphalt binder by means of a nozzle. The steam and asphalt combine in the throat of the nozzle orifice creating the foam.

Two types of foam can be produced; a "discrete foam" which is in the form of separate small bubbles as it leaves the nozzle, and "concentrated foam" in which the bubbles of the foamed binder are joined together. The steam is used to create the foamed binder and partly to control the character of the foam by varying the quantity and pressure.

It was reported (7) that using "concentrated foam" the aggregate temperature could be lower than with "discrete foam" and that with comparable asphalt contents, higher stability was achieved with foamed asphalt than with atomized asphalt. In work on standard specification asphalt concrete, it was found that aggregate temperature could be considerably lower using foamed asphalt.

Field experience. In 1956 two small test sections were lain in Iowa using in place stabilization on a layer of cinders and heavy clayey soil and six percent foamed asphalt. In 1957 (8) a larger

half mile section was laid. The soil was a clayey silt with a plastic index of 10 percent and a liquid limit of 20 percent. Five to six percent of foamed asphalt was used in the "concentrated" form. The roads have performed well but because of scuffing, seal coats had to be applied.

The city of Dubuque, Iowa (9) has used foamed asphalt in a variety of forms. A plant mix was used consisting of heated agricultural limestone and 10 percent foamed asphalt. An asphalt cement slurry was used consisting of crushed limestone, up to 12 percent clay, 13 percent foamed asphalt and enough water to produce the desired consistency (usually about 25 percent). A stock pile mix was produced from a heated limestone and six percent foamed asphalt. This mix was cooled by spraying water on it and is then stored in piles. Foamed asphalt was also used to pre-coat aggregate used for seal coat chips. It was found that less asphalt and aggregate are thus required. All work done in Dubuque showed no signs of wear six months after completion.

In Arizona (10) a highly porous volcanic ash was stabilized using 8.5 percent foamed asphalt and 10 percent moisture. Considerable saving was possible because about five percent less binder was necessary and the necessity of drying the aggregate was eliminated.

A fine sand was stabilized in place to provide a parking lot in Minnesota. (11) A three inch depth was stabilized using 4.5 percent of 200 penetration asphalt. No information is available as to

its performance.

In October, 1961, attempts were made to stabilize a highly plastic Manitoba clay with a natural moisture content of 30 percent, by the use of foamed asphalt. (12, 13) The greatest difficulty encountered with this material was to break the lumps down to small sizes. No reports are available as to this material's performance.

In summary it can be seen that foamed asphalt is a relatively new process, having been developed within the past ten years. Little published material on the basic properties of mixtures prepared with foamed asphalt or on design criteria is available. The long term behavior of foamed asphalt projects has not been evaluated because the short period of time since its development and because of the few projects upon which it has been used. Hence, it remains to be seen whether it will become a generally accepted process.

Stripping Tests to Indicate Asphalt Adhesion.

Another problem encountered with asphalt-aggregate systems is the tendency of the asphalt film to strip from the aggregate when exposed to water.[a] A number of tests, such as the Dow test (15), have been developed in an attempt to predict whether the asphalt will strip from the aggregate. Basically these tests consist of mixing a portion of the larger aggregate particles with asphalt, subjecting the mix to water and visually observing the effect. These tests usually indicate whether a mix will tend to strip but do not give any

[a] Reference 14 is an annotated bibliography on the subject.

indication of the quantitative effect of stripping on the physical properties of the pavement.

In 1942 a study was undertaken by the Public Roads Administration (16) to compare a sound paving aggregate with one which had performed badly when exposed to water. In order to try to eliminate some of the shortcomings of other stripping tests, the compressive strength of compacted specimens of the two mixes was measured both in a dry state and after immersion. The results of the tests were believed significant in that:

a. The test utilized the whole aggregate instead of one fraction.

b. Soundness of the aggregate was determined directly in terms of retention of stability.

c. Results were expressed in numerical values, unaffected by the personal factor involved in the estimation systems as used in other stripping tests, and

d. The results were consistent with the service behavior of the materials used.

A second part of this study was an investigation into the use of various mineral fillers. It was found that limestone dust was better than a clay filler and that a reduction in strength resulted when the clay was present as a coating on the aggregate.

Another investigation into the effect of additives was conducted (17) using the immersion-compression test, as well as other stripping tests. It was concluded that this test provides a useful

measure of the benefits to be derived from the use of additives.

In 1943 Erchma and Loomis (18) used the immersion-compression test to investigate the water resistance of bituminous mixes as affected by aggregate type, aggregate gradation, air voids, time of immersion, binder content, binder consistency, age of mix and aggregate degradation during compaction.

It was found that water susceptibility could be reduced by:

a. Reducing the air voids by improving the gradation of the aggregate.

b. Aging the mixture.

c. Increasing the binder consistency.

d. Reducing aggregate degradation during compaction.

A "Water Susceptibility Constant" defined as the rate of change of the strength reciprocal, was developed to give a quantitative numerical evaluation of the water resistance of a bituminous mix.

In 1947 a test procedure was outlined standardizing the procedure for the immersion-compression test. (19) This revised procedure was essentially that adopted tentatively by the ASTM in 1949 and as a standard test in 1960.

In 1948 Pauls and Goode (20) attempted to increase the sensitivity of the test. Tests were done to evaluate the effect of the use of a vacuum process to accelerate saturation of the specimens and higher immersion temperatures and longer immersion periods.

Vacuum saturation of the specimens before immersion gave

results which were inconsistent with field experience and was not used further. Immersion conditions reported to give results consistent with field performance were one day at 140°F or four days at 120°F for hot mixtures and one day at 100°F for cold mixtures.

Goldbeck (21) compared paving mixtures using the immersion compression test and a laboratory traffic test. He obtained poor correlation between the two tests and suggested that the immersion compression test be made more severe.

A design procedure for surfacing mixtures has been set forth by Goode (22) based on the immersion compression test. Stability and durability are assured by specifying the minimum dry compressive strength and percent of retained strength permissable. The minimum allowable dry strength was 200 psi and the minimum allowable strength retained after immersion was 70 percent (140 psi soaked strength). Flexibility and anti-skid properties are allowed for by setting the asphalt content at that amount which produces six percent air voids.

The immersion compression test appears to be a cheap, easy method of determining the effect of water on bituminous mixtures. It undoubtedly has many faults but the results can be numerically expressed and reproduced within reasonable limits.

CHAPTER III

MATERIALS

Aggregate

The aggregate used was a crushed, well graded gravel with about 25 percent passing the No.200 sieve. The source was the Christianson Pit, used by the Alberta Department of Highways, located about 40 miles north of Calgary on the east side of Highway 2 near the town of Carstairs. Classification of the aggregate as sampled according to the Unified Soil Classification System (23) was GM-GC and according to the AASHO (24) Designation M145-49 was A-2-4.

The aggregate samples were taken from a stockpile, placed in sacks and brought to the laboratory. Here it was placed in large pans and allowed to dry for about one week before being processed.

Table I shows a summary of the results of the classification tests on the aggregate, the sieve analyses of the two gradations used and AASHO (25) specifications for untreated base course material, Designation M147-55, grading D. Detailed results are contained in Appendix A.

This material is rated by the Unified System as a good material for subgrades and base courses when not subjected to frost action. Due to its poor drainage characteristics it is rated as a poor base directly under bituminous pavements. A petrographic analysis performed in accordance with Ontario Department of Highways procedure (26), indicates that according to their standards, its use

should be restricted to the construction of shoulders or mulch pavements.

Examination of Table I indicates that gradation A lies outside the gradation limits specified for base course material and that gradation B lies within the specifications.

TABLE I

RESULTS OF CLASSIFICATION TESTS ON AGGREGATE

AASHO Designation M145-49	A-2-4
Unified Systems Soil Type	GM-GC
ASTM Apparent Specific Gravity (retained on No.10 sieve)	2.72
ASTM Bulk Specific Gravity	2.56
Water Absorption	2.0%
Uniformity Coefficient	720
Liquid Limit (pass No.40 sieve)	24.0
Plastic Limit	18.1
Plastic Index	5.9
Petrographic Number	174
Los Angeles Abrasion Test - loss	25%

WASHED SIEVE ANALYSIS

U.S. Sieve No.	Percent of Material Passing			
	As Sampled	Gradation A	Gradation B	AASHO (25) Specifications for Base Course Material
1 inch	100	100	100	100
¾ inch	100	100	100	-
3/8	84	-	-	85 - 50
No.4	71	50	46	65 - 35
No.10	57	35	26	50 - 25
No.40	41	26	17	30 - 15
No.100	32	-	-	-
No.200	27	18	12	15 - 5

Asphalt

The asphalts used were a 150-200 penetration asphalt cement and an MC 3 cutback, supplied by Husky Oil and Refining Company

Limited of Lloydminster, Alberta. The analyses shown in Table II and Table III were performed by the supplier. The asphalt was received and stored in five gallon cans. When asphalt was required it was removed from the can. Any asphalt remaining after a day's work was discarded rather than returning it to the can.

TABLE II

ANALYSIS OF 150-200 PENETRATION ASPHALT CEMENT

Specific Gravity at 60°F	1.0291
API Gravity at 60°F	6.0
Flash Point (C.O.C.)	425+°F
Penetration at 77°F (100 gms., 5 sec)	153
Ductility at 77°F (5 cm./min.)	100+
Loss on heating at 325°F (50 gm., 5 hours)	1% -
Penetration after L.O.H. test, percent of original Penetration	70%+
Soluble in Ccl4	99.8+

TABLE III

ANALYSIS OF MC 3 CUTBACK ASPHALT

Specific Gravity at 60°F	0.9820
API Gravity at 60°F	12.6
Flash Point (T.O.C.)	150+°F
Water Percent	Nil
Saybolt Furol Viscosity at 140°F	442 seconds

Distillation

	Percent of total over	Percent of distillate total Over
437°F	0	0
500°F	2.5	12.5
600°F	13.0	65.0
680°F	20.0	100.0

Residue to 680°F volume by difference	80.0%
Residue penetration at 77°F (100 gms.,5 secs.)	189
Oliensis spot test (15% Xylene)	Negative
Soluble in Ccl4	99.8+%
Ductility at 77°F (5 cm per minute	100+ cm.

Water

In order to eliminate any effects of impurities in the water used for soaking, it was decided to use distilled water.

CHAPTER IV

TESTING PROGRAM

Outline of Testing Program

The testing program was divided into five main tasks. They were:

a. Preparation of the aggregate.

b. Mixing and aeration.

c. Compaction of the mixture.

d. Curing the specimens.

e. Testing the specimens.

The aggregate was dry sieved into four fractions in the laboratory and then recombined in the desired batch size to give two different gradations: gradations A and B listed in Table I.

Four different mix types were produced in the following manner:

a. Cold Mix - air dried aggregate at room temperature mixed with an MC-3 cutback asphalt in the university pugmill, designated by the letter C.

b. Hot Mix - aggregate at 325°F mixed with asphalt cement in the university pugmill, designated by the letter H.

c. Cold Foam Mix - aggregate containing moisture was mixed with foamed asphalt cement in the B.C.H.[a] pugmill, designated by the letters C.F.

d. Hot Foam Mix - aggregate at 325°F mixed with foam asphalt cement in the B.C.H. pugmill, designated by the letters HF.

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[a] Bernard, Curtis, Hoggan
Engineering and Testing Limited

Each of the two gradations were mixed with arbitrarily selected asphalt contents of three, five and seven percent by each of the four above listed processes. By use of a numerical suffix for the asphalt content and letters for gradation and mix type, a complete designation of each batch of six samples was available. Thus, HA5 designates a hot mix of gradation A mixed by the university pugmill with five percent asphalt cement. Six samples were prepared from each mixture and a total of 144 specimens tested.

Compaction was by double-plunger static compression of 3000 psi held for two minutes according to ASTM Designation D1074-60 (1). The cold mixes were compacted at room temperature while the hot mixes were at 260°F. After compaction, the samples were cured for 24 hours at 140°F.

After cooling, three of the six specimens were tested in unconfined compression. The other three were soaked for four days at 120°F, checked for swelling and water absorption and then tested in unconfined compression.

Preparation of Aggregate

The tendency of an aggregate with a wide range of particle sizes to segregate during handling was recognized early in the program. To minimize this, it was decided to separate the entire sample (about 800 lbs) into fractions so that by blending, small samples could be obtained with reasonable certainty as to its gradation.

The air dry aggregate was separated into four fractions as

follows:

a. Passing the $\frac{3}{4}$ inch sieve and retained on the No.4 sieve.

b. Passing the No.4 sieve and retained on the No.10 sieve.

c. Passing the No.10 sieve and retained on the No.40 sieve.

d. Passing the No.40 sieve.

The sieving was done on a mechanical sieve shaker. About 20 lbs of air dry aggregate was dumped in and sieved for two to three minutes. The material on the sieves was removed and dumped into large pans for storing. The procedure was repeated until all the aggregate was processed.

When sieving was completed a sample was selected from each pan by the method of quartering and a washed sieve analysis performed on it. The average results of these analyses are shown in Appendix A.

Two different aggregate gradations were chosen; one approximating the gravel as it was taken from the stockpile and falling outside the AASHO (25) specifications in the region of the No.100 sieve (gradation A); the other falling well within specifications for base course materials (gradationB). By a method of trial and error, calculations (27) the desired gradations were approximated by blending the four sieved fractions. The proportions used are shown in Appendix A. It was found that a batch of 12 kilograms was adequate to form six specimens.

Mixing and Aeration

This phase included placing the aggregate in the pugmill, introducing the asphalt, mixing, removing from the pugmill and storing or aeration until compaction was begun. Since each mixing procedure has some differences, they will be described under separate headings.

A short description of the two pugmills is given in Appendix B.

Cold Mix. Twelve kilograms of air dry aggregate at room temperature was dumped into the pugmill and mixed for about five seconds to ensure uniform distribution of the aggregate. The desired amount of MC-3 cutback asphalt was weighed out in a container and heated on a small hot plate. When it was at 150°F the mixer was turned on and the asphalt slowly poured in. The total mixing time was 120 seconds of which 30 seconds was used to pour in the asphalt. The mixer was stopped, the bottom of the pugmill opened and the mix scooped into two large baking pans for aeration. Aeration consisted of heating the mixture in a loose condition for 18 hours at 100°F (19). After aeration, the mixture was allowed to cool to room temperature before compaction. The exact weight of asphalt used was determined by weighing the empty asphalt container and subtracting this weight from the weight of the asphalt plus container determined before heating.

Hot Mix. The aggregate and pugmill were both heated overnight; the aggregate to 325°F in an oven and the pugmill, with heaters at the maximum setting (about 250°F). A quantity of 150 - 200 penetration asphalt cement was heated on a hot plate. When the asphalt was at 325°F the aggregate was transferred to the pugmill and mixed for a few seconds to ensure uniformity. The mixer was then switched on and the asphalt slowly poured in. Pouring the asphalt took about 30 seconds of the total mixing time of 90 seconds. The mixer was stopped and the mixture removed from the pugmill,

scooped into two large pans and immediately placed into an oven pre-heated to 280°F. The exact quantity of asphalt used was determined in the same manner as for the MC-3 cutback asphalt.

Cold Foam Mix. The aggregate weighed out in six 28 pound batches, was transported to the B.C.H. laboratory in plastic bags. Here it was mixed by hand with eight percent moisture, the amount required to produce maximum dry density in the Standard Proctor Compaction test. A quantity of 150-200 penetration asphalt cement was heated to 325°F and poured into the electrically heated asphalt container where it was maintained at this temperature. The float was placed in the liquid asphalt, allowed to settle and a pencil mark put opposite the float pointer on the scale. The wet mix was then dumped into the mixing unit and about five seconds mixing is done to ensure a uniform aggregate. Next the steam was turned on for a few seconds to blow out any asphalt left in the nozzle, and the asphalt pump turned on. The mixer was then started and the valves controlling the steam and the asphalt opened. When the float pointer reached the mark set out on the scale, both valves were quickly closed. A total mixing time of 60 seconds was used, 10 seconds of which was used to introduce the asphalt. When the mixing cycle was completed, a small door on the bottom was opened and the mixture was caught in the large tray. The mixer was then stopped, the mixture scooped back into the plastic bags and the bags sealed. The distance that the float actually moved was measured and the asphalt content was corrected. The bags were transported to the university and allowed to sit for 24 hours before compaction.

Hot Foam Mix. The soil was transported in plastic bags to the B.C.H. laboratory. The pugmill and the 28 lb batches were both heated overnight; the aggregate to 325° in an oven and the pugmill to 280°F. When the pre-heated 150-200 penetration asphalt had been placed in the heated container the aggregate was dumped into the mixer and about five seconds mixing done to ensure a uniform aggregate. The method of asphalt metering, mixing and removal from the pugmill was exactly the same as for the cold foam mix. The aggregate was scooped from the tray into two large baking tins, placed in a portable oven pre-heated to 300°F and transported to the university laboratory for compaction.

Compaction of the Mixture.

Both hot mixes were compacted at 260°F, while both cold mixes were at room temperature. The hot mixes were maintained at 280°F by storing them in an oven. The mold, plungers and spatula were preheated in near-boiling water. The molds and plungers were wiped off with a clean cloth and covered with a light film of oil. Then the bottom plunger was put in place with the molding cylinder supported temporarily on the two steel bars.

The amount estimated to produce a specimen four inches high and four inches in diameter was weighed out on a portable balance. About one-half of this mixture was placed in the mold and spaded with a spatula. Fifteen of the blows were delivered around the perimeter to reduce honeycomb and the remaining ten at random over the mixture. The other half of the mix was then quickly poured in and spaded in a similar way.

The top plunger was then inserted and the mixture compressed under an initial load of 150 psi to set the mixture against the sides of the mold. The support bars were removed and a 3000 psi load applied and held for two minutes by the Tinius Olsen testing machine.

The sample was ejected by pushing through the top plunger on a hydraulic press. The sample height was checked so that an adjustment in the amount of material could be made if necessary. When six samples of one asphalt content had been molded, they were placed in the oven for curing.

Curing the Specimens.

All molded samples were cured in the same manner. When a set of six specimens had been molded they were placed on a large baking tin and placed in an oven pre-heated to 140°F for 24 hours.

Testing the Specimens.

After the specimens had cured they were allowed to cool to room temperature for at least two hours. The bulk specific gravity of each specimen was obtained by dividing the oven dry weight by the bulk volume of the specimen. The bulk volume was obtained by subtracting the immersed weight from the surface dry weight of the specimen. The surface dry weight was obtained by allowing the specimen to soak in water until no bubbles were given off, drying the surface with a towel and weighing. At the same time the height of the specimen was measured and recorded. All weights were measured to the tenth of a gram.

The set of six specimens was divided into two groups so that

the average bulk specific gravity of each group was essentially the same. One group of three was then set on a glass plate and immersed in the constant-temperature water bath and maintained at 120°F for four days.

After four days they were removed to a second water bath maintained at room temperature and stored there for two hours. They were then removed from the water and the immersed weight and the surface dry weights measured and recorded.

The specimen was seated on a spherically seated bearing block and placed in the Tinius Olsen testing machine. The upper testing head was brought almost into contact with the upper surface of the specimen and an Ames dial was installed to check the rate of compression. With the machine set to its lowest range[a], the specimen was broken at a strain rate of 0.2 inches per minute. The maximum load as registered by the pointer was recorded.

The other group of three specimens, having been cooled to room temperature for at least four hours after curing, were failed in the same manner. For these specimens it was necessary to set the machine on medium range[b].

Discussion of Procedure and Apparatus.

Procedure. The choice of the four fractions into which the soil was sieved was mainly a choice of convenience. In order to permit closer control over the blended gradation, more sieves should have been used and the fractions should have been sieved longer, especially

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a The lowest range was 0 to 600 kg with the smallest division equal to one kg.

b Medium range was from 0 to 3000 kg, with the smallest division equal to five kg.

the smaller sizes. The No.4 and passing No.40 fractions were larger than the other two. However, no 3/8 inch sieve was available and it was felt that the time required to sieve the passing No.40 fraction into smaller fractions would be prohibitive.

The batch size of 12 kilogram, enough for six samples, is larger than that specified by the procedure outlined in ASTM Designation D1074-60 (1). It specifies that one specimen shall be mixed at a time. It was decided that this method was too time consuming and that the pugmills would not efficiently mix such a small batch. However, by ensuring the selection of large representative samples from the sieved fractions and by care in mixing and compaction, it was felt that the effects of segregation were minimized.

Compaction by the double plunger action worked quite well. Cleaning and oiling the plungers and mold after each usage ensured that there was no binding while compacting and extruding samples.

It was found by trial and error that in order to ensure that the mixture was at 260°F when compacted, it was necessary to maintain it in the oven at 280°F. This allowed ample time to place the mixture in the mold, spade it and place the top plunger in position.

The hot foam mixes, for economic reasons, were mixed three at a time and stored in an oven at 280°F until compacted. Compaction was begun immediately and completed as quickly as possible. Despite this, the last mix to be compacted was held at 280° for about three hours. This may have caused some of the asphalt to be oxidized.

It was impossible to obtain the proper specimen height without weighing out the portions first. Even by this procedure it usually took two or three adjustments before the correct size was obtained.

In order to minimize segregation, the mix was spread onto the loading scoop as uniformly as possible, and then scraped into the mold. However, the spading action seemed to shake the fine material to the bottom, leaving the large particles on the top. This gave a smooth bottom surface and a pitted top surface on the specimen. The spading action did, however, aid in reducing the honeycomb on the sides of the specimens.

This method of compaction appeared to eliminate the weakness caused by the horizontal layers present in the hammer type compaction. Since it was realized that different degrees and types of compaction produce different densities and strengths, care was taken to ensure that the compaction procedure was varied as little as possible.

The cutback asphalt mix was aerated for 18 hours before compaction in an attempt to simulate aeration which occurs in the field before compaction (19). The cold foam mix was stored for 24 hours in the plastic bags in order to minimize the effect of a six hour time difference between the time of compaction of the first and last batches. It was felt that if the time lapse between mixing and compaction for the first and last batches were 24 and 30 hours respectively, instead of one and seven hours, that any effect of a time differential upon the two mixtures would be greatly reduced. The

plastic bags appeared to minimize moisture loss from the mix since moisture condensed on the inside of the bags upon cooling and was still present the following day. This condensate was returned to the mixture by kneading the bags until it disappeared.

All samples were cured for 24 hours at 140°F after compaction. In the case of the cutback mixture it is said that this brings about more uniformity in strength by reducing the effect of variations in the degree of curing that occurs during laboratory storage (20). The volatile content of the hot and hot foam mixes was small while that of the cold foam was larger. However, it was decided to cure all mixes in the same manner to simplify comparison.

In the surface dry weight determinations water was still being taken on after one minute immersion. Hence the immersed weight kept increasing until air bubbles ceased to be given off. Consequently, the immersion time for surface dry weight determinations was increased to a period long enough to ensure no more air bubbles were given off.

Immersion at 120°F for four days has been shown to provide results consistent with service behavior for hot mixes in the North Eastern United States (20). These immersion conditions were extended to include all mixtures. It may have been unduly severe for the cutback mixture since the recommended immersion temperature for cold mixes is 100°F (19). It was assumed, however, that by the 18 and 24 hour curing periods most of the volatiles had been removed, leaving a specimen cemented by a penetration grade asphalt.

The strain rate was specified as 0.05 inches per minute per inch of height of the specimen. The strain rate varied as the specimen height enabling a comparison of results of different size specimens. The total strain applied to the samples before failure was not measured.

Equipment. It was not feasible to assemble a foaming nozzle on the university pugmill in time for use in this thesis. This necessitated the use of two different pugmills, hence introducing several additional variables to the mixing operation. Two entirely different mixing actions produce the most important difference. Other differences are the duration of mixing, method of measuring the quantity of asphalt in and size and speed of the paddles.

Indications were that some of the large aggregate particles were being crushed between the paddles and the sides of the pugmill during mixing. To what extent this altered the gradation of the aggregate is unknown.

The Tinius Olsen machine held the static load for compaction quite well. After the load was fully on there was no problem in keeping the load constant to within about one percent.

Maintaining the rate of strain constant was more difficult. At one valve setting the strain rate decreased slowly as the load was applied and increased rapidly after the sample had failed. The valve setting had to be changed continuously to keep the strain rate at 0.2 inches per minute.

Photo 1. Forming Mold and Plungers

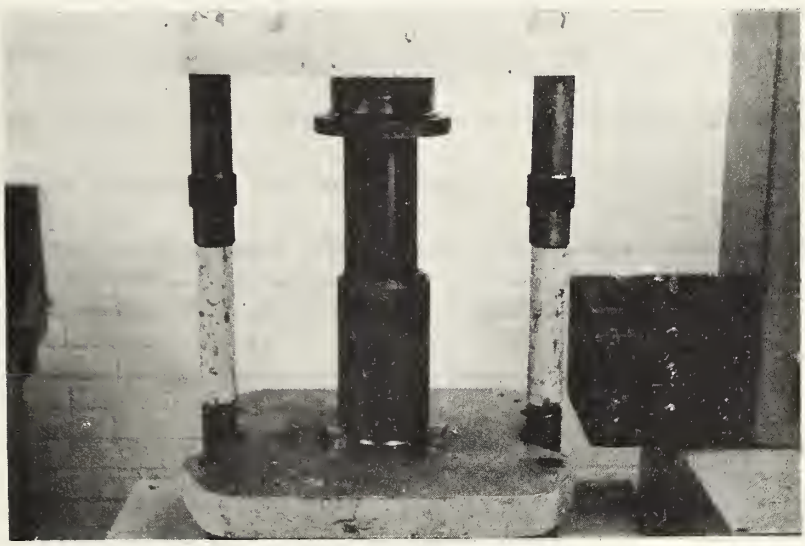

Photo 2. Specimen Ready For Compaction

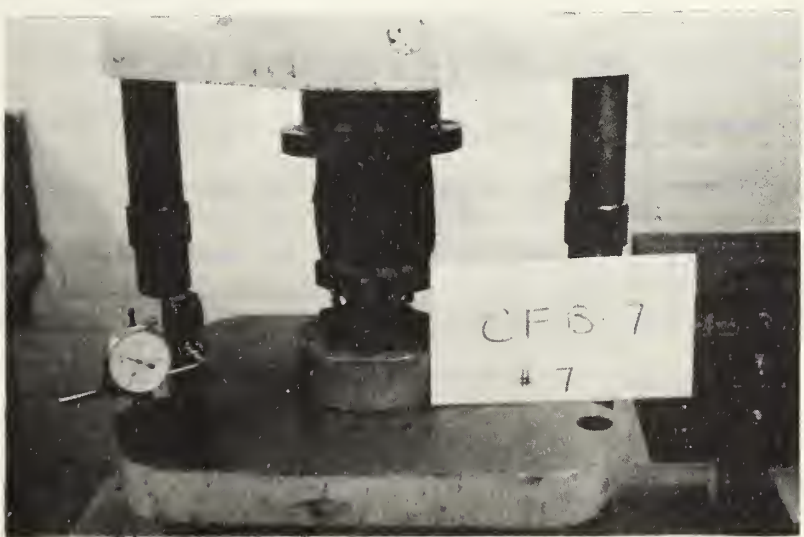

Photo 3. Testing in Unconfined Compression

CHAPTER V

TEST RESULTS

Summary of Results

Table IV contains a summary of all the test results and the calculations performed. Each value for strength, swell and absorption is the average of the results of three specimens. All other values are the average results of tests performed on six specimens.

Binder contents listed in Table IV are based on the residual asphalt in the mixtures. When the cutback asphalt was used it was assumed that the volatiles were driven off by aeration and curing. The contents of MC3 used were 3.0, 5.0 and 7.0 percent with residual amounts of 2.4, 4.0 and 5.6 percent. When asphalt cement was used it was assumed that there was no volatile loss and hence no reduction in asphalt content due to curing.

Values for some quantities are missing either because the sample was in no condition for handling or because they were not measured for the first few samples.

The volume calculations in Table IV were performed in two different manners; by using the apparent specific gravity of the aggregate and by using the bulk specific gravity when an allowance for asphalt absorption.

McLeod (28) recommends the use of the bulk specific gravity to calculate volume relationships in paving mixtures of absorptive aggregates. By use of this concept void volumes do not include internal void space as it would if the apparent specific gravity were used. This approach also recognizes that part of the

TABLE IV

SUMMARY OF TEST RESULTS

1 Asphalt Number Content %	2 Mix Type	3 Dry Density of Total Mix pcf	4 Std. Dev. of Dry Density pcf	5 Dry Strength psi	6 Std. Dev. of Dry Strength psi	7 Soaked Strength psi	8 Std. Dev. of Soaked Strength psi	9 % of Strength Retained	10 % Volumetric Swell	11 % Water Absorbed	12 % Voids (a)**	12 % Voids (b)***	13 % Volume Occupied by Aggregate (a)**	13 % Volume Occupied by Aggregate (b)***	14 % Voids in Mineral Aggregate (a)**	14 % Voids in Mineral Aggregate (b)***	15 % Volume of Voids Occupied by Bitumen (a)**	15 % Volume of Voids Occupied by Bitumen (b)***
2.4	CA3	139.9	0.3	38	7	*	-	*	*		16.3	13.0	76.6	84.5	20.4	15.5	24.6	17.1
4.0	CA5	143.1	0.2	157	13	2	1	1	11.7	7.4	9.9	7.5	81.2	88.3	18.8	11.7	47.7	45.4
5.6	CA7	146.2	0.2	129	13	8	1	6	10.6	6.6	6.0	3.8	81.3	86.3	18.7	13.7	67.6	72.8
2.4	CB3	139.2	0.2	124	18	*	-	*	*		15.0	11.	78.9	85.4	21.1	14.6	25.8	27.1
4.0	CB5	145.5	0.1	161	4	4	1	3	10.3	7.9	8.6	5.5	82.3	87.4	17.7	12.6	51.3	56.6
5.6	CB7	145.5	0.2	96	6	8	1	8	8.9	5.4	6.2	3.8	81.2	86.3	18.8	13.7	66.8	71.9
3.1	HA3	138.9	0.7	299	36	35	7	12			16.4	11.9	78.1	82.9	21.9	17.1	28.0	31.3
4.8	HA5	139.9	0.8	561	33	61	13	9			11.8	8.4	78.6	83.4	21.4	16.6	46.7	48.2
7.1	HA7	143.8	0.6	570	60	117	24	21			6.3	3.9	79.3	84.1	20.7	15.9	69.9	76.5
3.1	HB3	138.1	0.2	339	50	23	3	7		5.1	15.0	11.3	78.8	83.8	21.2	16.2	29.3	31.2
5.1	HB5	139.3	0.3	393	36	45	3	12		5.0	11.6	8.5	78.2	82.9	21.8	17.1	47.1	50.1
7.0	HB7	140.6	0.2	299	8	89	15	30		4.4	8.4	6.2	77.7	82.3	22.3	17.7	63.5	67.1
3.8	CFA3	140.6	0.3	318	31	19	1	6	2.4	6.0	13.9	11.3	78.4	83.2	21.6	16.8	35.7	31.5
5.2	CFA5	138.7	0.2	219	14	17	3	8	1.7	5.5	13.6	11.0	76.2	81.8	23.8	19.2	43.8	42.2
7.1	CFA7	134.9	0.4	151	8	14	1	9	1.5	5.6	13.1	11.3	73.1	77.7	26.9	22.3	51.3	50.5
3.2	CFB3	140.6	0.3	305	7	24	1	8	1.5	5.3	14.7	10.9	78.8	83.6	21.2	16.4	30.9	33.4
5.1	CFB5	136.2	0.2	229	6	19	1	8	0.9	5.4	14.5	11.5	75.3	80.0	24.7	20.0	41.4	42.4
7.2	CFB7	134.9	0.2	175	11	14	1	8	0.8	5.4	13.0	10.8	73.1	77.7	26.9	22.3	51.7	51.6
3.3	HFA3	138.7	0.4	403	50	16	4	4	*	7.4	14.7	9.9	79.4	83.9	20.6	16.1	33.6	37.4
5.3	HFA5	142.4	0.4	649	39	48	11	7	7.4	5.9	9.0	5.9	79.8	84.7	20.2	15.3	55.7	62.7
7.5	HFA7	145.5	0.2	590	58	144	25	24	4.4	3.8	4.3	1.9	79.8	84.8	20.2	15.2	78.7	86.7
3.1	HFB3	138.3	0.3	244	37	31	3	13	8.7	7.4	14.8	12.3	78.9	83.6	21.1	16.4	30.8	24.6
5.1	HFB5	140.6	0.2	464	43	42	1	10	7.0	5.7	10.7	8.4	78.8	83.7	21.2	16.3	50.1	49.3
7.1	HFB7	143.2	0.3	422	35	46	8	11	7.	5.4	6.8	4.4	78.5	83.3	21.5	16.7	68.5	73.6

*Condition of sample prevented a value from being obtained.
**Value obtained by using ASTM apparent specific gravity.
***Value obtained by using ASTM bulk specific gravity.

bitumen is absorbed into the aggregate and that only a portion of the total bitumen content remains as a coating on the aggregate particle.

Goode (22) proposed the use of the effective specific gravity for use in mix design. He states that the use of the apparent specific gravity will lead to bleeding pavement while the bulk specific gravity will result in a lean mixture subject to abrasion.

It was felt that by using both specific gravities the extreme range of values would be covered. The true value would be likely to lie between the two extremes. The value of asphalt absorption is assumed to be 60 percent of the absorption of water as determined in the standard ASTM test, Designation C127-42 (29). This assumption appears reasonable according to published information on the estimation of asphalt absorption. (28)

The average values only of each quantity are plotted. In order to obtain an idea of the variation in values, the standard deviation from the mean strength and density has been tabulated in Table IV.

The standard deviation is a constant deviation from a central value of a particular set of values. It may be used to predict the probability of obtaining a subsequent measurement within a defined range. Thus it is said to indicate the confidence which one may have in those measurements (31). Once the standard deviation has been determined it is then possible to calculate the standard deviation of the mean of the group of measurements.

The mean value of a group measurement may be expressed with 90 percent confidence as

$$s \pm \frac{\sigma}{N} \times 1.645$$

in which s = mean value of the group of measurements

σ = standard deviation

N = number of measurements

Thus, for example, the dry strength of mix type HA7 may be expressed with 90 percent certainty as 570 ± 57 psi.

For each relationship a maximum of three points are available to be plotted graphically. With such a few points there was no justification to drawing smooth curves through the points except in the case of the density/asphalt content relationship where it is probable that a curved relationship exists. In all other cases the points were joined by straight lines.

Since the residual binder content was one of the variable factors for each mix type, it was plotted against the following quantities:

a. Dry density of total mix for gradation A in Figure 1.

b. Dry density of total mix for gradation B in Figure 2.

c. Percent voids in the mineral aggregate[a] for all mixes in Figures 3 and 4.

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a Defined as the total volume of the intergranular void space between the aggregate particles divided by the volume of the specimen. In soil mechanics it is termed the porosity.

d. Percent water absorbed[a] for all mixes except HA in Figure 5.

e. Percent swell[b], for those mixes for which swell was measured, in Figure 6.

f. Compressive strength of the cold mix for both gradations in Figure 8.

g. Compressive strength of the hot mixes for both gradations in Figure 9.

h. Compressive strength of the cold foam mixes for both gradations in Figure 10.

i. Compressive strength of hot foam mixes for both gradations in Figure 11.

In Figure 7 an attempt is made to correlate swell and water absorption for the various mix types as percent swell is plotted against the percent moisture absorbed.

The dry compressive strength is plotted versus the cement/void ratio[c] to logarithmic scales in Figures 12 and 13. The two graphs are necessitated because of the slightly different values of the cement/void ratio caused by using the two different specific gravity values. Figures 14 and 15 are the plots of the soaked compressive strengths versus the cement/void ratio on logarithmic scales.

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a Calculated on the basis of weight increase of specimen divided by the original weight of dry aggregate in the specimen.

b Calculated on the basis of volume increase divided by the original specimen volume.

c In this thesis the cement/void ratio is defined as the volume of the asphalt divided by the total volume of voids in the mineral aggregate (i.e. the volume of asphalt plus air voids). This is the value of column 15 of Table IV expressed as a decimal.

Discussion of Results

Aggregate. The sieve analysis on the aggregate as sampled indicates that its gradation would normally preclude its use as a base course material. Examination of Table I shows that its gradation lies almost entirely outside the AASHO limits for untreated granular base course material. (25)

Listed in Table V are some Ontario Department of Highways (30) specifications for coarse aggregates produced from wayside material for use in base course construction. The corresponding properties of the aggregate used in this thesis are listed for comparison. An examination of this table indicates that the coarse aggregate is of marginal quality.

TABLE V

PHYSICAL REQUIREMENTS FOR COARSE AGGREGATES

Property	Maximum allowable Value	Test Results
[a] Absorption - percent by weight	2.0	2.0
[b] Petrographic Number	160	174
[a] Los Angeles Abrasion - percent loss	35	25

a Performed in accordance with ASTM specifications.

b Performed in accordance with Ontario Department of Highways procedure outlined in Appendix C.

The plasticity properties of the fine aggregate are also marginal. The test value of the liquid limit is only one percent below the maximum allowable value of 6 percent.

Examination of the sieve analyses of gradations A and B in Table I indicates that both are dense graded. Dense graded aggregates tend to have a large number of points of contact between individual particles, resulting in high strengths when confined.

Various grading curves for maximum density have been proposed, based on the formula:

$$P = 100 \left[\frac{d}{D}\right]^n$$

in which

P = percentage of material by weight which passes a given sieve having openings of width d

D = the maximum particle size of a given aggregate

n = an exponent for which several different values have been proposed.

In Appendix A, gradations A and B are compared to Fullers maximum density curve for which the exponent n equals $\frac{1}{2}$. Gradation B appears to most closely approximate the maximum density curve but has less intermediate sizes and tends to be more open graded than gradation A.

The percentages passing the No.200 sieve are 18 and 12 for gradations A and B respectively, as compared to 6 percent for maximum theoretical density. It is said that in practice maximum densities are produced by gradations having an excess of small sizes as compared to the theoretical amount for maximum density. This, plus the more uniform distribution of particle sizes may explain the higher densities of gradation A in the standard Proctor compaction test.

The gradations used were blended from the four sieve fractions on the basis of washed sieve analyses as illustrated in Appendix A. A considerable portion of the fine aggregate is not actually present in the mix as free material but is coated on the larger particles. For example, 5.7 percent of the total amount of 17.9 percent of the material passing the No.200 sieve in gradation A is present as a coating on the material coarser than the No.4 sieve. This will cause gradation A to behave similar to an aggregate with only 12.2 percent passing the No.200 sieve, assuming that none of the fine aggregate is dislodged during blending, mixing and compaction.

Density. Figures 1 and 2 illustrate that the density of gradation A mixes were generally about one to three pcf higher than similar ones of gradation B. This is not unexpected because the standard Proctor test indicated the same result. Compared at the same binder content, the mix types listed in general order of decreasing density produced were cold mix, hot foam, hot and cold foam mixes.

The cold mix, hot mix and hot foam mix were increasing in density with increasing binder content. With one exception, they have not reached maximum density at the highest asphalt content employed. Since asphalt is not as effective a lubricating agent as water, it is probable that more asphalt than water will be required to produce maximum density. Thus, it is probable that more than eight percent binder would be required to produce the maximum density of the total mix. The density of the cold foam mix was

BINDER CONTENT VS. DRY DENSITY OF TOTAL MIX

FOR GRADATION A

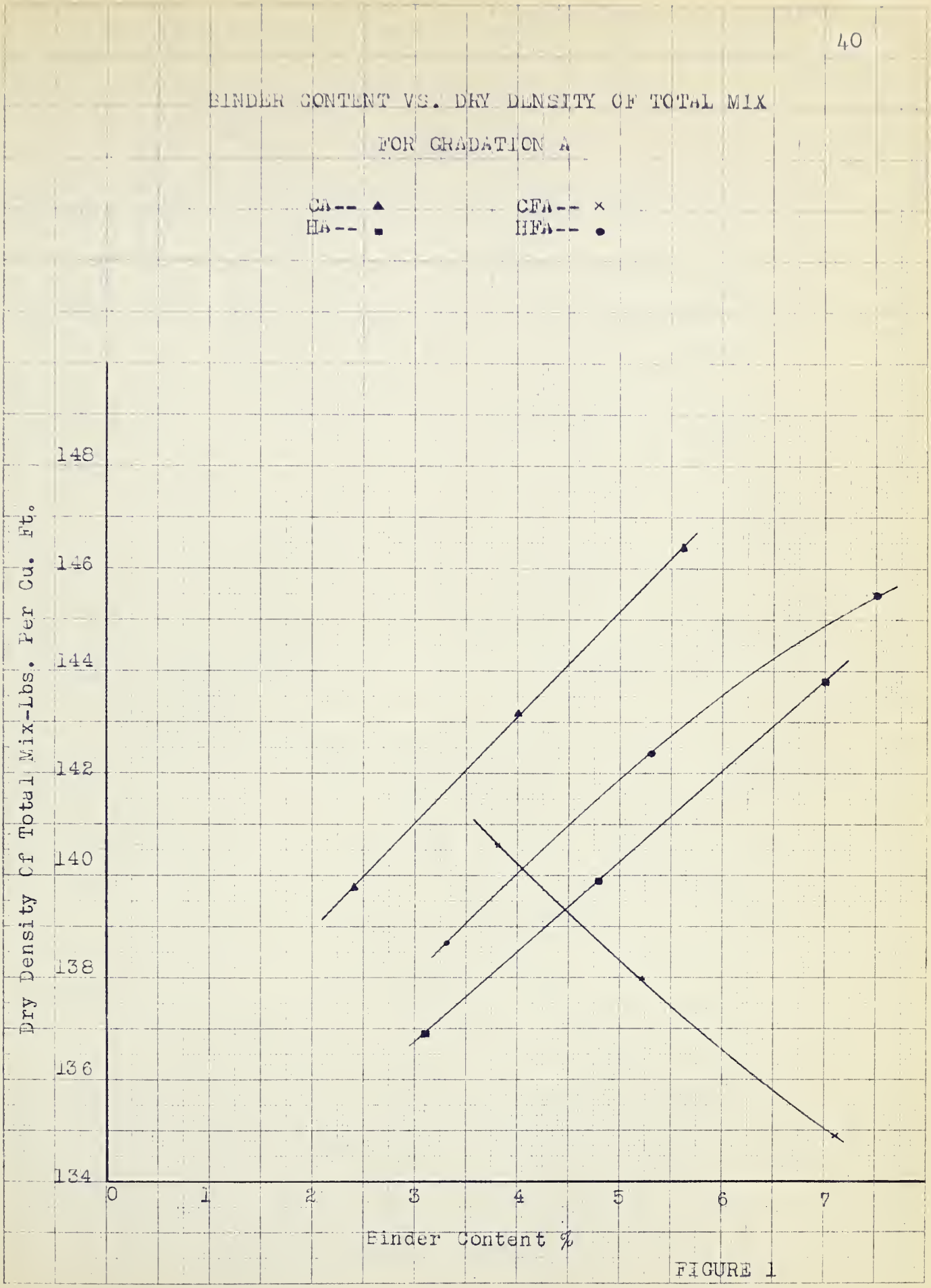

FIGURE 1

BINDER CONTENT VS. DRY DENSITY OF TOTAL MIX

FOR GRADATION B

CB-- △ CFB-- ⊗
HB-- □ HFB-- ○

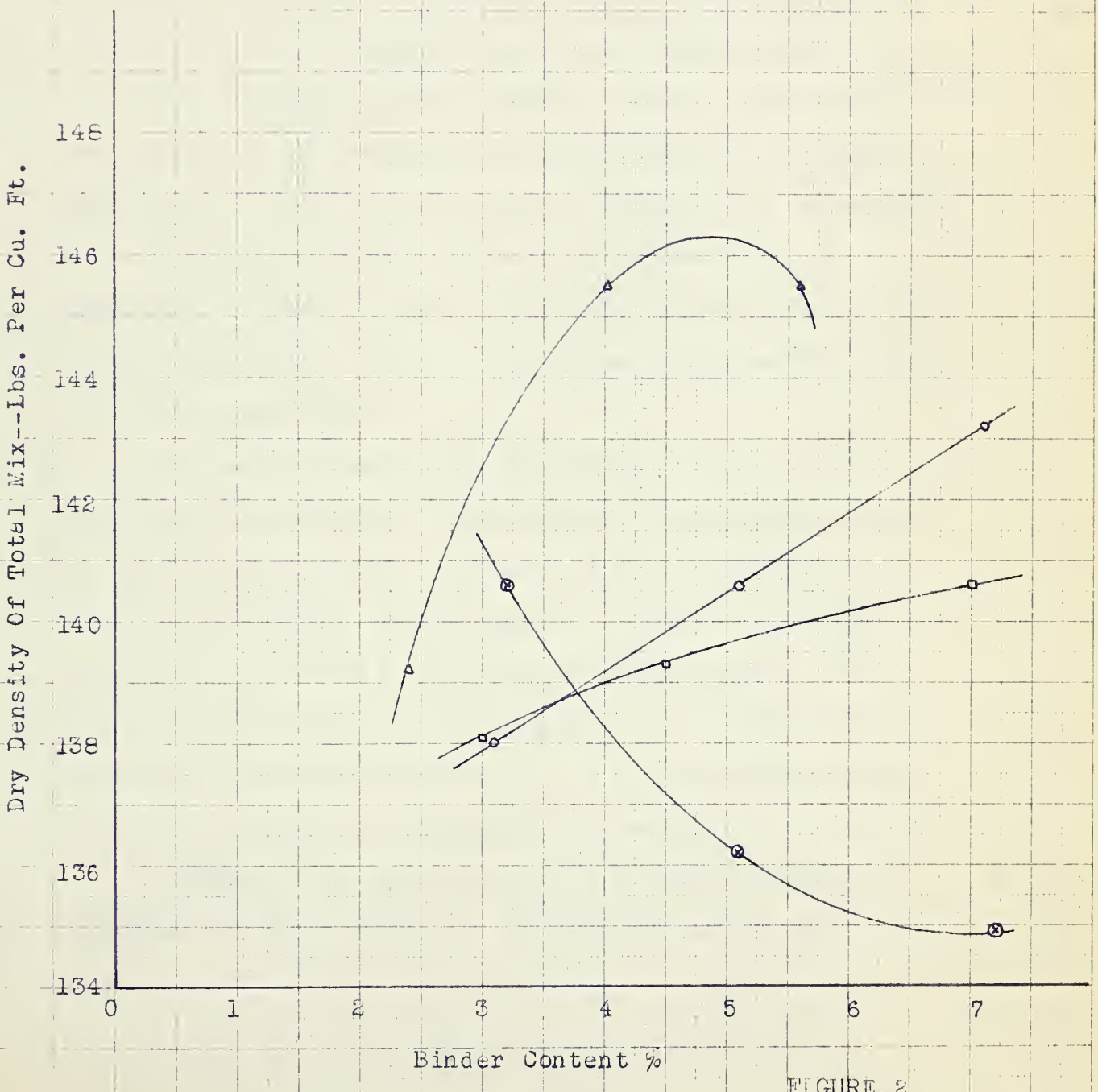

FIGURE 2

decreasing with increasing binder content. The aggregate was at the optimum moisture content when mixed with the binder. Thus, when the binder is added, the densities decrease as they would if the aggregate were compacted above the optimum moisture content.

The reason for a change in the dry density of the total mix with a change in the binder content may be determined by examining two factors; the change in density of the aggregate portion of the mix and the change in weight of the binder in the mixture.

The voids in the mineral aggregate, or VMA, refers to the total volume of the intergranular void space between the aggregate particles. As such, it can be used as an indication of the density of the aggregate particles contained in a mixture. Thus, an examination of the variation of the VMA with binder content in Figures 3 and 4 will furnish an explanation for the density changes noted in Figures 1 and 2.

For example it can be seen from Figures 3 and 4 that for gradation B of the hot mix the minimum VMA, or the maximum density of aggregate in the mixture, was achieved at 3.1 percent binder. At 5.1 percent binder, the VMA increased slightly, indicating a decrease in the density of the aggregate in the mixture. The actual increase in total density is probably due to the increase in the weight of the asphalt from 3.1 to 5.1 percent less the slight decrease in weight of the aggregate in the mixture.

The highest density achieved was at 5.6 percent binder of gradation A of the cold mix. At this point the density of

BINDER CONTENT VS. PERCENT VOIDS* IN

THE MINERAL AGGREGATE

*-Calculated using bulk specific gravity

HFA-- • HFB-- ○
CFA-- × CFB-- ⊗
HA-- ■ HB-- □
CA-- ▲ CB-- △

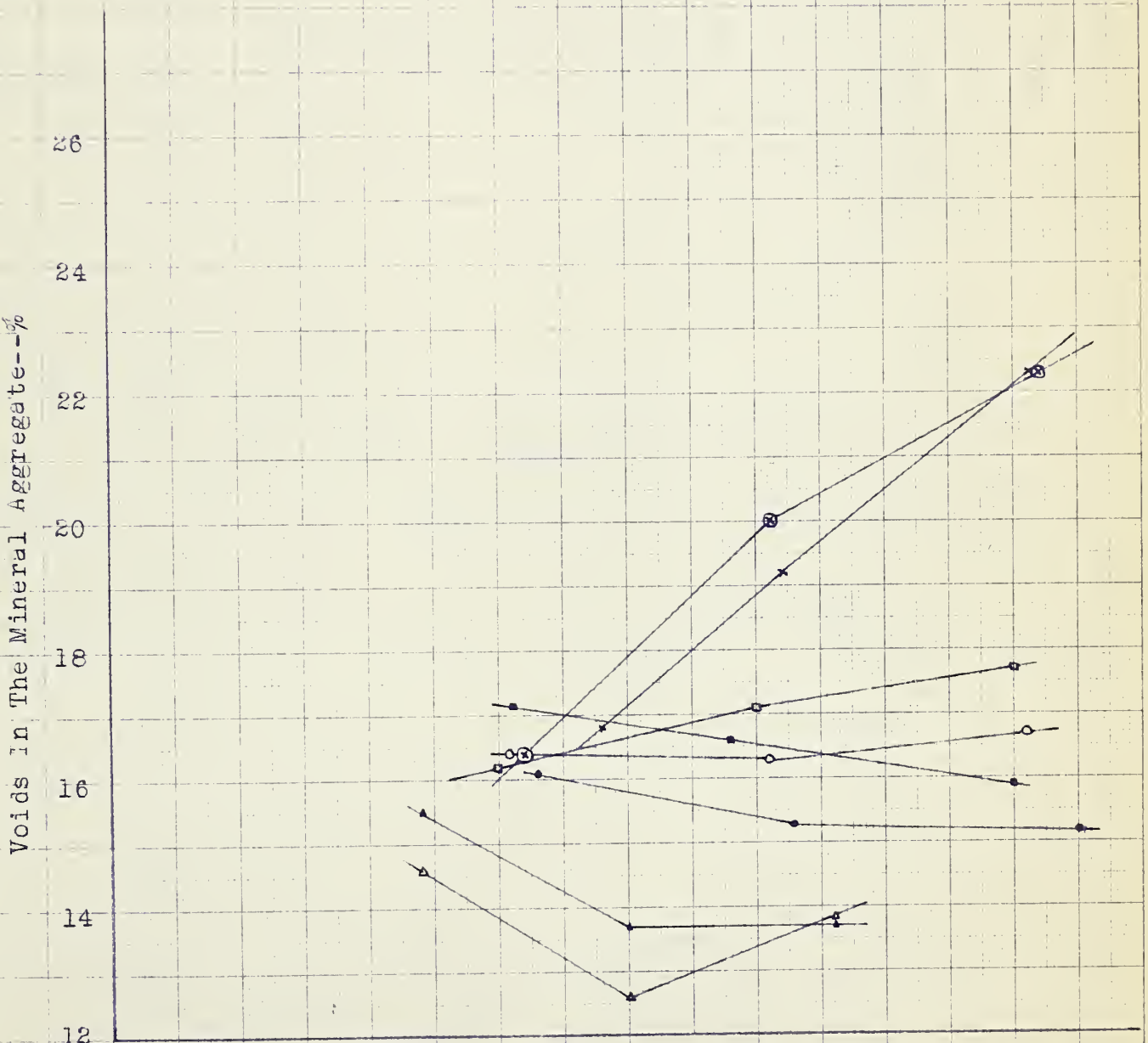

FIGURE 3

BINDER CONTENT VS. PERCENT VOIDS* IN

THE MINERAL AGGREGATE

*- Calculated using apparent specific gravity

HFA-- • HFB-- ○

CFA-- × CFB-- ⊗

HA-- ■ HB-- □

CA-- ▲ CB-- △

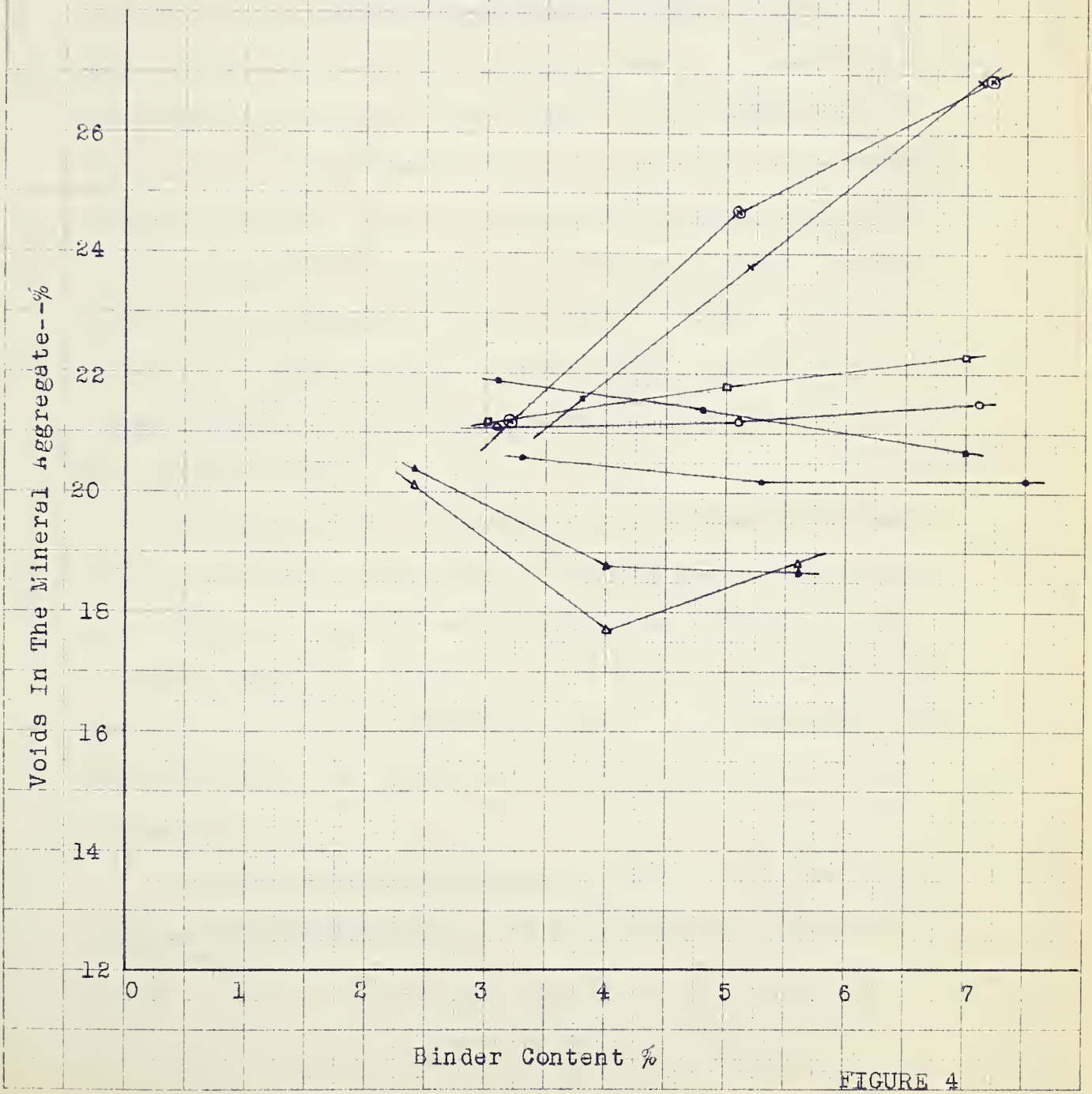

FIGURE 4

gradation A appeared to be increasing while gradation B appeared to have passed through a maximum at about 4.8 percent binder.

This was the only mix exhibiting a maximum density and an optimum binder content. Although asphalt is not thought to be as effective a lubricating agent as water, the optimum asphalt content for compaction was about three percent lower than the optimum moisture content. This points out the possibility of the curve for cold mix gradation B in Figure 2 being in error. The relatively low standard deviation for this mix type in Table IV indicates that an error in measurement sufficient to change the shape of the curve is not likely. Since this was one of the first mixes tested a slight change in the procedure of determining the bulk specific gravity of the specimens is more probable. A slightly lower density at 4.0 percent binder or a slightly higher one at 5.6 percent binder would produce a curve in agreement with the trends shown by the other mix types.

The density of both gradations of the hot foam mix increased through the range of binder used, with gradation B increasing at a slower rate and producing lower densities than gradation A. The densities produced are higher than those for the hot mix, indicating that the foamed asphalt remains more fluid than asphalt cement at the same temperature, thus providing the better lubrication for compaction.

The density of both gradations of the hot mix increased with increasing binder content, but again gradation B increased at

a slower rate. Minimum values for VMA were obtained at about seven percent binder for gradation A and three percent for gradation B. Thus, gradation A produced densities higher than gradation B at seven percent binder, while at three percent binder gradation B produced the higher density.

The minimum VMA value and maximum density of the total mix for both gradations of the cold foam mix were achieved at about three percent binder. As the binder content increased, the density of the total mix and the density of the aggregate in the mix decreased. The addition of fluid asphalt to the aggregate already at the optimum moisture content resulted in compaction samples at or near saturation. Evidence of this was the brownish liquid which was squeezed out at the bottom of the mold when the static load was applied.

For example, sample 2 of CFB3 contained 117.3 grams of water upon compaction. According to calculations performed using the apparent specific gravity there were 121.6 cc of voids available for this water, indicating a degree of saturation of 96 percent. With increasing binder content the volume of unfilled voids decreases, thus increasing the degree of saturation of the compacted samples.

Water absorption and swell. The mechanism by which water penetrates soil or bituminous mixtures is not well understood. An extensive review of existing theories on flow of moisture through partially saturated soils under pressure and electric potentials and thermal gradients has been prepared by Harihanan (32). According to Hutcheon (33), when a sample of soil is subjected to a

temperature gradient, the flow of moisture from warm to cold regions occurs largely in the vapor phase at first. The condensation of the excess vapors in the cooler regions results in a flow of moisture in the liquid phase from the cold to the warm regions, once a favourable pressure gradient has been established. When the soil is sufficiently moist to permit active liquid flow, a state of equilibrium cannot be reached and a continuous circulation of water takes place.

The flow of moisture in soils takes place through the natural capillary tubes. The aggregate particles in bituminous mixtures are covered with an asphalt film which coalesce with adjoining films when the mixture is compacted. Thus, bituminous mixtures are relatively impervious because most of the void spaces are not interconnected. In order for moisture to gain entry, it seems likely that the specimen must expand or swell to enable passages to be developed between the isolated void spaces. Hence, it is possible that for a given amount of moisture absorption, considerably more swelling will occur in a bituminous mixture than in the untreated soil; since no experimental evidence was available, this was impossible to prove.

Figure 5 provides an indication of the relative effectiveness of the binder in preventing the entrance of moisture into samples. The general tendency is for the most water to be absorbed at low binder contents and the least at high binder contents. The hot foam mixes and the cold mixes exhibit the greatest reduction in absorbed water, with increased binder content. The hot mixes and

BINDER CONTENT VS. PERCENT WATER ABSORBED

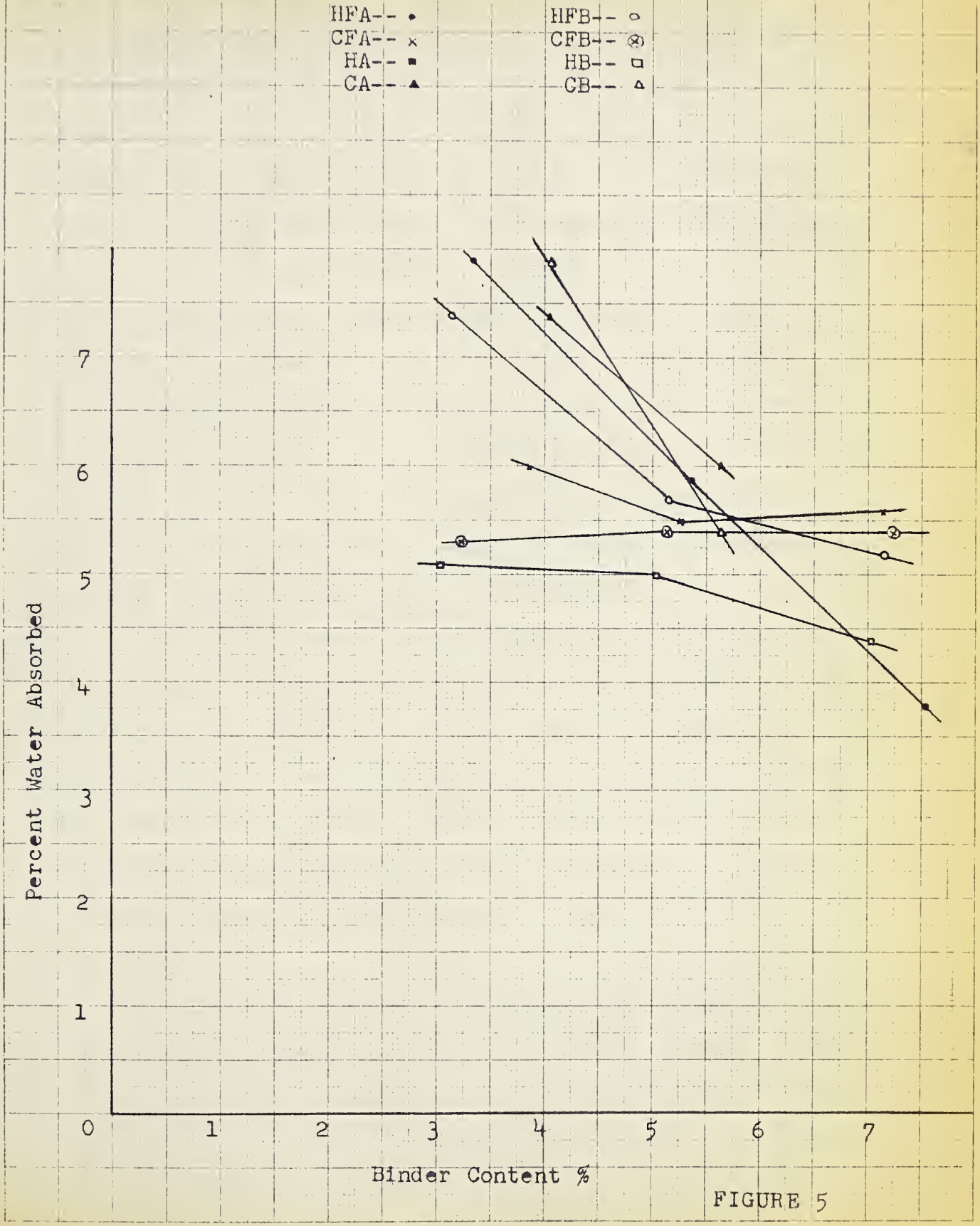

FIGURE 5

cold foam mixes showed little change in absorbed water with changes in the binder content.

Thus, an increase in the binder content has been quite beneficial in waterproofing the cold mix and the hot foam mix. On the other hand, it appears to have had very little effect on the water absorbed by the hot mix and the cold foam mix. A possible explanation for the water absorption and swelling characteristics exhibited will be set forth at the end of this section.

Krchma and Loomis (18) found that water absorption was at a minimum and strengths at a maximum at an optimum binder content. Water absorption increased and strengths decreased with an increase or decrease in binder content. No satisfactory explanation could be proposed to explain this. No optimum binder content for water absorption was found in this investigation, although it might exist at binder contents higher than those investigated.

It was also reported that water absorption was decreased by decreasing the void volume. Evidence from this investigation in support of this is contained in Table IV. However, sometimes large reductions in void volumes are necessary to obtain small reductions in absorbed water. For example, reducing the percent voids from 14.8 to 6.8 by increasing the binder content, has resulted in a reduction of absorbed water from 7.4 to 5.2 percent for the hot foam mix, gradation B.

From Figure 6 it can be seen that in general, the swell of gradation A mixes was greater than the same mix type of gradation B. The swell of the cold foam mixes was considerably lower than that of the

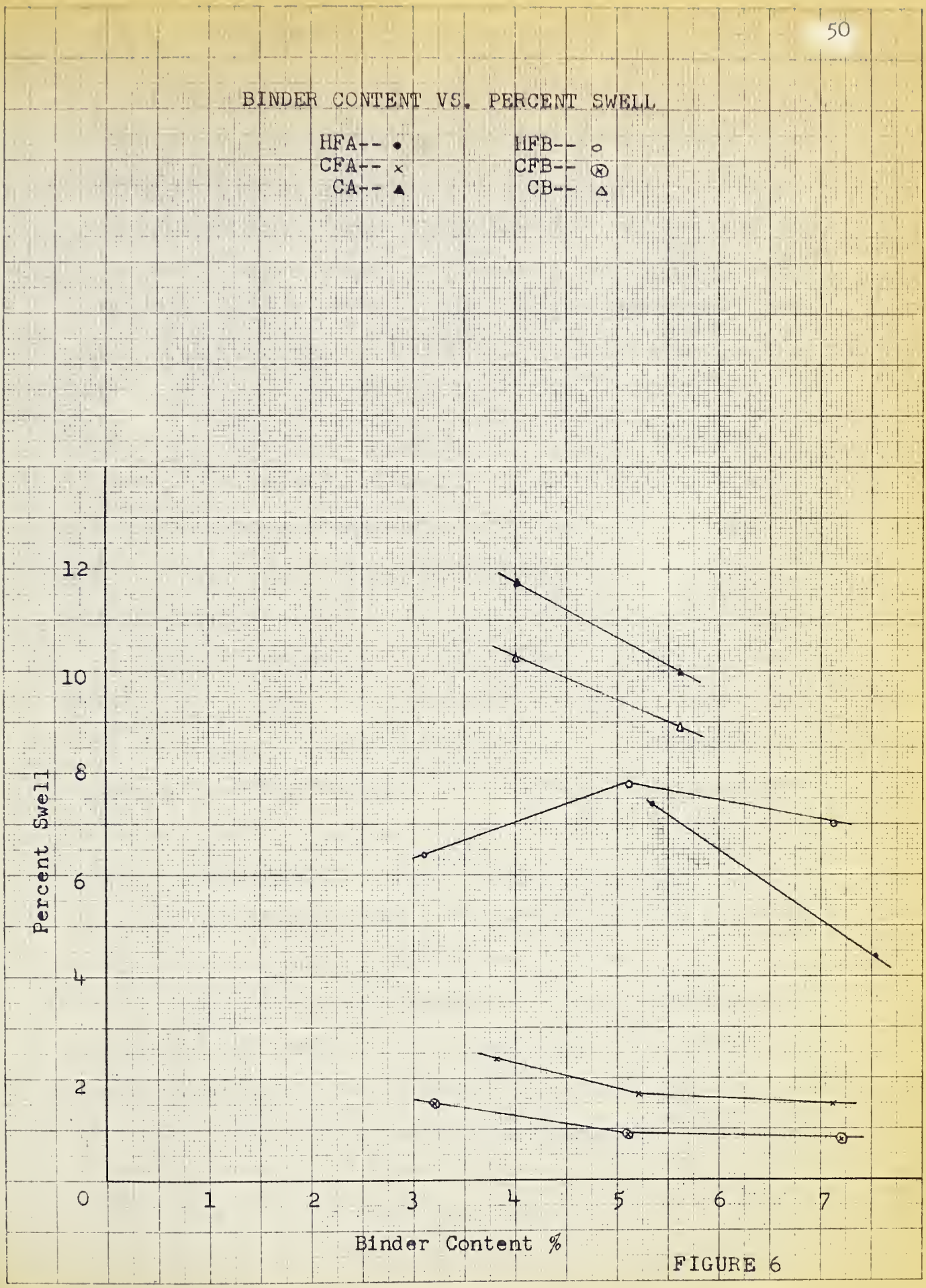
BINDER CONTENT VS. PERCENT SWELL
HFA-- •
CFA-- ×
CA-- ▲
HFB-- ○
CFB-- ⊗
CB-- △
Percent Swell
0
2
4
6
8
10
12
0
1
2
3
4
5
6
7
Binder Content %

FIGURE 6

other mix types. Increasing the binder content resulted in little further reduction for the cold foam mixes but appeared to cause considerable reduction in the swell of the other mix types.

Further trends are difficult to determine from Figure 6 because of the limited data available. For example, it was not known whether swell of cold mixes and hot foam mixes at lower binder contents was greater or less than swell at five percent. The fact that more water was absorbed at three percent than at five percent binder indicates that at three percent swell also might have been higher. However, at this low binder content, the mixes would likely be more permeable and would have more void space available than at higher binder contents. Hence, when water is absorbed, they may not swell as much as indicated in Figure 6 by gradation B of the hot foam mix.

Figure 7 illustrates how the samples react volumetrically to the absorption of water. The relationship of swell to absorbed water was assumed to be a linear one. This is in agreement with Endersby (2) who found that swell was substantially proportional to water absorption where the same soil and the same curing methods were used. From Figure 7, it can be seen that in general, for a given amount of water absorption, the cold mixes swell the most, the hot foam mixes next and the cold foam mixes the least.

All cold foam mixes were compacted at a moisture content such that the compacted samples were near saturation. Most of this moisture was evaporated from the samples during the 24 hour curing period leaving void spaces inter-connected by the paths through which

PERCENT SWELL VS. PERCENT WATER ABSORBED

HFA-- • HFB-- o
CFA-- × CFB-- ⊗
CA-- ▲ CB-- △

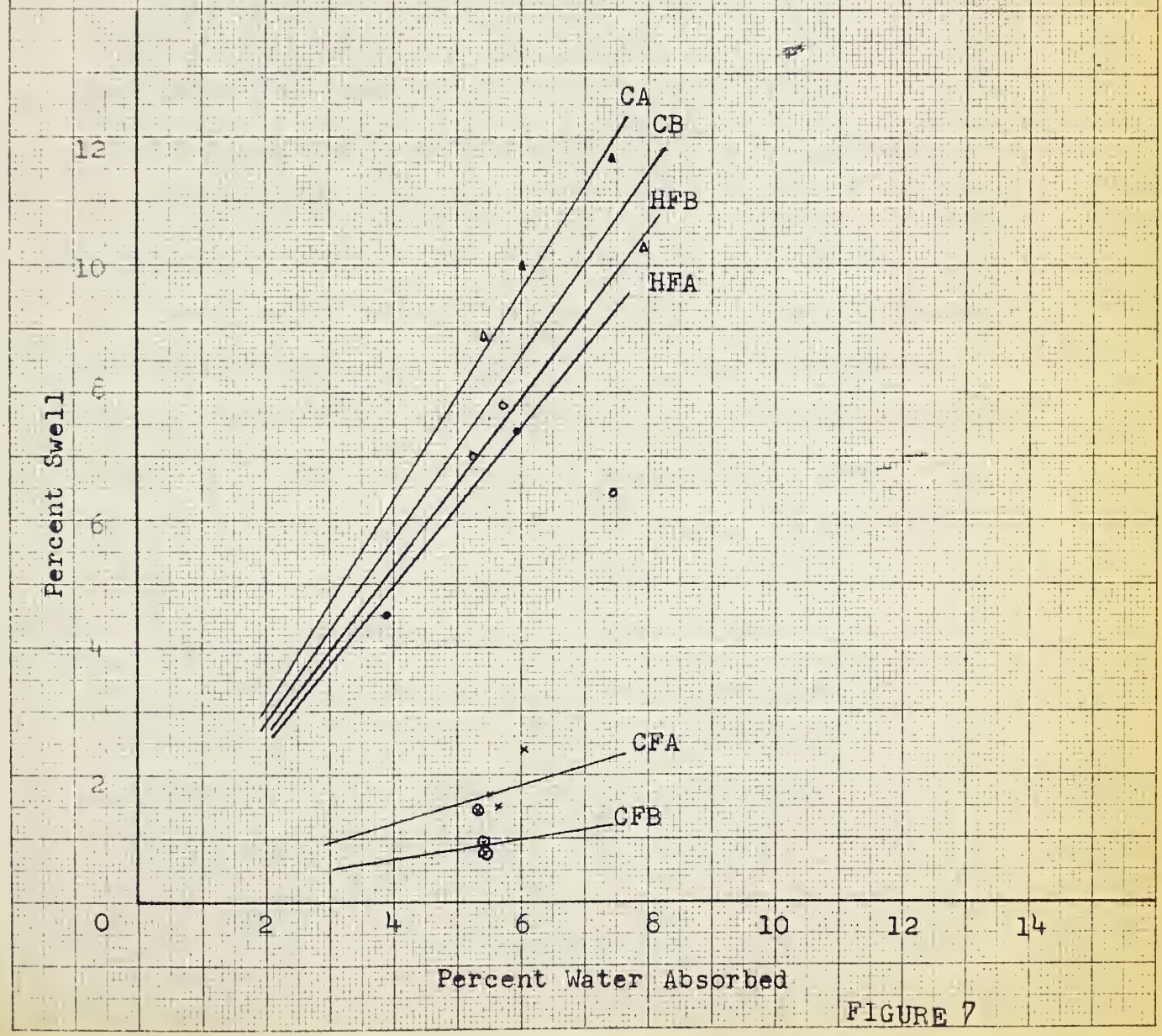

FIGURE 7

the steam escaped. Table IV shows that the void space in the samples remained essentially constant throughout the range of binder contents employed. As shown in Figure 6, the water could be absorbed easily through these paths into the inter-connected void spaces of the immersed specimens with little swelling. Since the space into which the water was absorbed remained essentially constant, the amount of water absorbed also varied little as shown in Figure 5.

Water absorption in the other mix types also appeared to be governed, at least partially, by the amount of void space available in the mixture for water. For example, by decreasing the void space from about 15 percent to 7 percent, the water absorption of gradation B of hot foam mix was decreased from about 7 percent to 5 percent.

The swelling characteristics exhibited by the hot, hot foam and cold mixes are much more difficult to explain. When swelling occurs it is possible that many of the asphalt-aggregate bonds are disrupted by the stripping action of the water. Thus it would be expected that more swell would occur in mixtures with weak asphalt-aggregate bonds. This seems to be borne out by the fact that in Figure 7 the hot foam mixes cemented by asphalt cement exhibit less swell than the cold mixes which are cemented by the cutback asphalt.

Strength. The unconfined compression test was selected as a cheap, easy test to perform and has been frequently used in stabilization investigations. The strength values were not corrected for area change during compression and it is probable

that end effects may have influenced the results. When absolute values are desired, these factors are important. However, when the results are viewed relatively or expressed in percentage of stability loss, these factors are not as important.

The unconfined compressive strength of a sample consists of two parts; first is the cohesion supplied by the adhesion of the binder to the aggregate and second is the friction between the aggregate particles. The cohesive strength is dependent upon the thickness of the binder film and the strength of the bond between the asphalt and the aggregate. For any particular aggregate the friction developed depends mainly on the aggregate density. In these tests it was impossible to detect what portion of the strength was produced by each source.

Examination of Table IV shows dry strengths ranging from 35 to 650 psi. Generally gradation A produces higher dry and soaked strengths than gradation B. Soaked strengths are from 0 to 30 percent of the dry strengths. The strengths produced by the cold mixes are shown graphically in Figure 8. From Table IV it can be seen that in general, the strengths produced by the cold mixes were the lowest of any mix type even though the density was highest.

Figure 8 shows that dry strengths about 30 percent lower were produced by gradation A when the mix was not aerated prior to molding. When the mixes were aerated they were not stirred or mixed in any way to increase exposure surface and increase the curing action. Higher strengths may have been obtained if the aeration had been improved or prolonged.

BINDER CONTENT VS. COMPRESSIVE STRENGTH

OF COLD MIXES

CA (non-aerated) -- ▼
CA (aerated) -- ▲
CB -- △

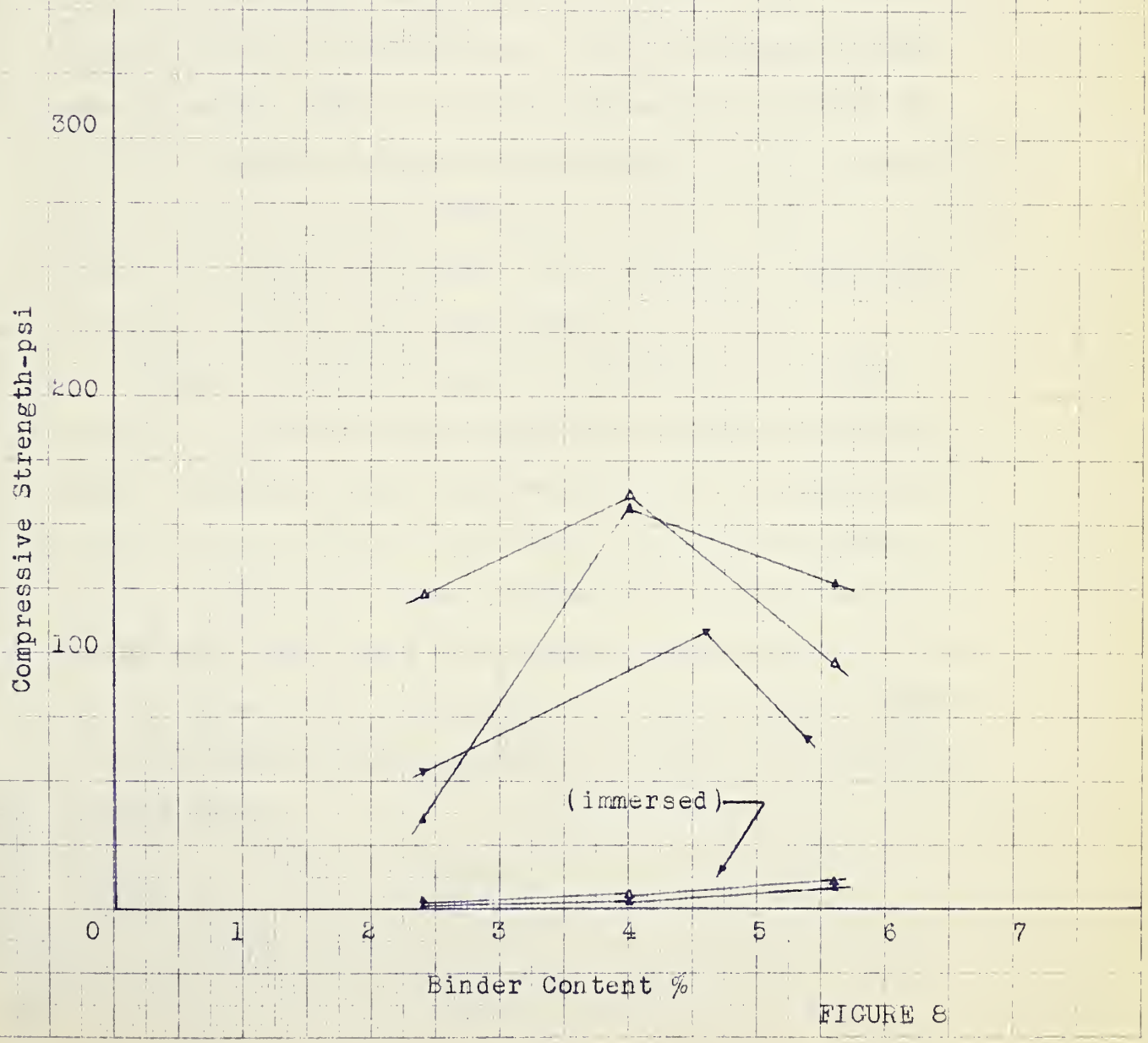

FIGURE 8

The probable reason for low immersed strengths of the cold mix was that the immersion condition was unduly severe when compared with procedures used by others. The recommended immersion temperature for cutback mixtures is 100°F instead of 120°F. A study by Pauls and Goode (20) showed that strengths using 100°F immersion temperatures were up to 35 percent higher than strengths using 120°F immersion. Identical immersion conditions, however, allow a comparison of all the mixtures under similar conditions.

Possibly higher strengths of the cold mixes would have been attained if moisture had been used to improve the asphalt dispersion in the mix. Pennell (34) found that maximum unconfined compressive strength in compacted sand mixtures occurred at a different moisture content for each cutback asphalt content used. The moisture content decreased as the asphalt content increased but was always sufficient to produce the maximum density of the mix.

Figure 9 shows that gradation A produced dry and soaked strengths of about 25 percent higher than gradation B in the hot mixes. Gradation A produced maximum soaked and dry strengths of about 570 psi and 120 psi, respectively, both at seven percent binder. Gradation B produced a maximum dry strength of about 400 psi at five percent binder and a maximum soaked strength of 90 Psi at seven percent binder. Increasing the binder content from three to seven percent increased the soaked strengths of both gradations about $3\frac{1}{2}$ times.

BINDER CONTENT VS. COMPRESSIVE STRENGTH

OF HOT MIXES

HA -- ■
HB -- □

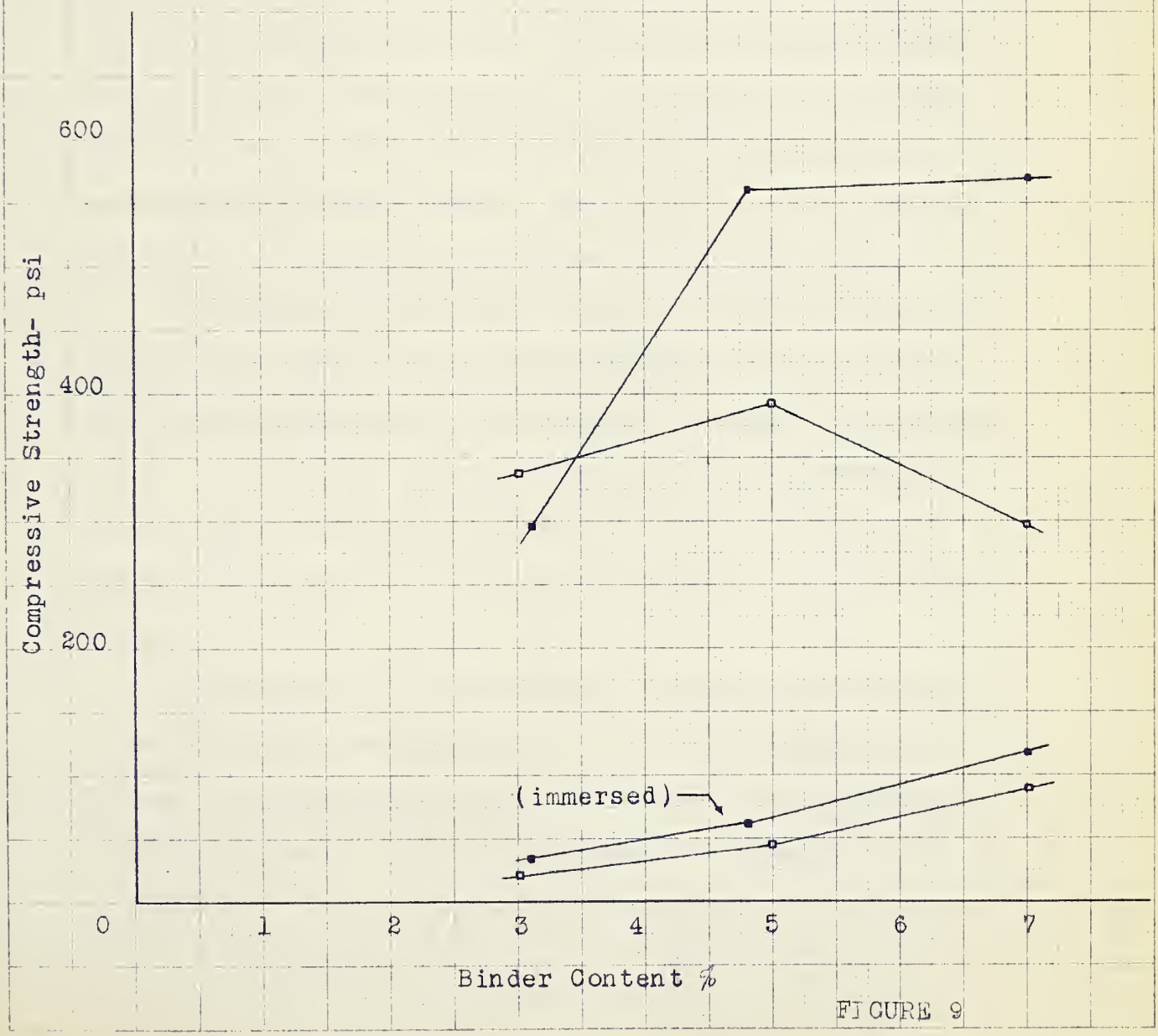

FIGURE 9

Figure 10 illustrates that gradation A again shows considerably higher strengths than gradation B. The maximum dry strength of gradation A is 650 psi at five percent binder while gradation B at seven percent binder has a maximum dry strength of 425 psi. Maximum soaked strength for both gradations occurred at seven percent binder. Gradation A has a maximum soaked strength of about 150 psi, more than twice the maximum soaked strength of gradation B.

In the three cases discussed, the maximum soaked strength and the minimum water absorbed occurred at seven percent binder. Although higher binder contents were not investigated it is likely that the soaked strengths would increase with binder content to an optimum value. At this binder content any increase in waterproofing due to increased binder content would be offset by a loss in cohesion and friction due to the increased thickness of the binder film.

By comparing Figure 10 with Figure 3, it can be seen that the maximum dry strength and minimum VMA generally coincided at about five percent binder. The aggregate was in its densest state and the friction component of strength was greatest. A large enough quantity of binder was available to coat all the particles; yet the binder film around the particles was thin enough to provide strong cohesion.

From Figure 11 it can be seen that the cold foam mixes show strength tendencies vastly different from the other types of mixes. Maximum dry and soaked strengths occur at three percent binder and minimum strengths at seven percent binder. The strengths of the

BINDER CONTENT VS. COMPRESSIVE STRENGTH

OF HOT FOAM MIXES

HFA-- ▪
HFB-- o

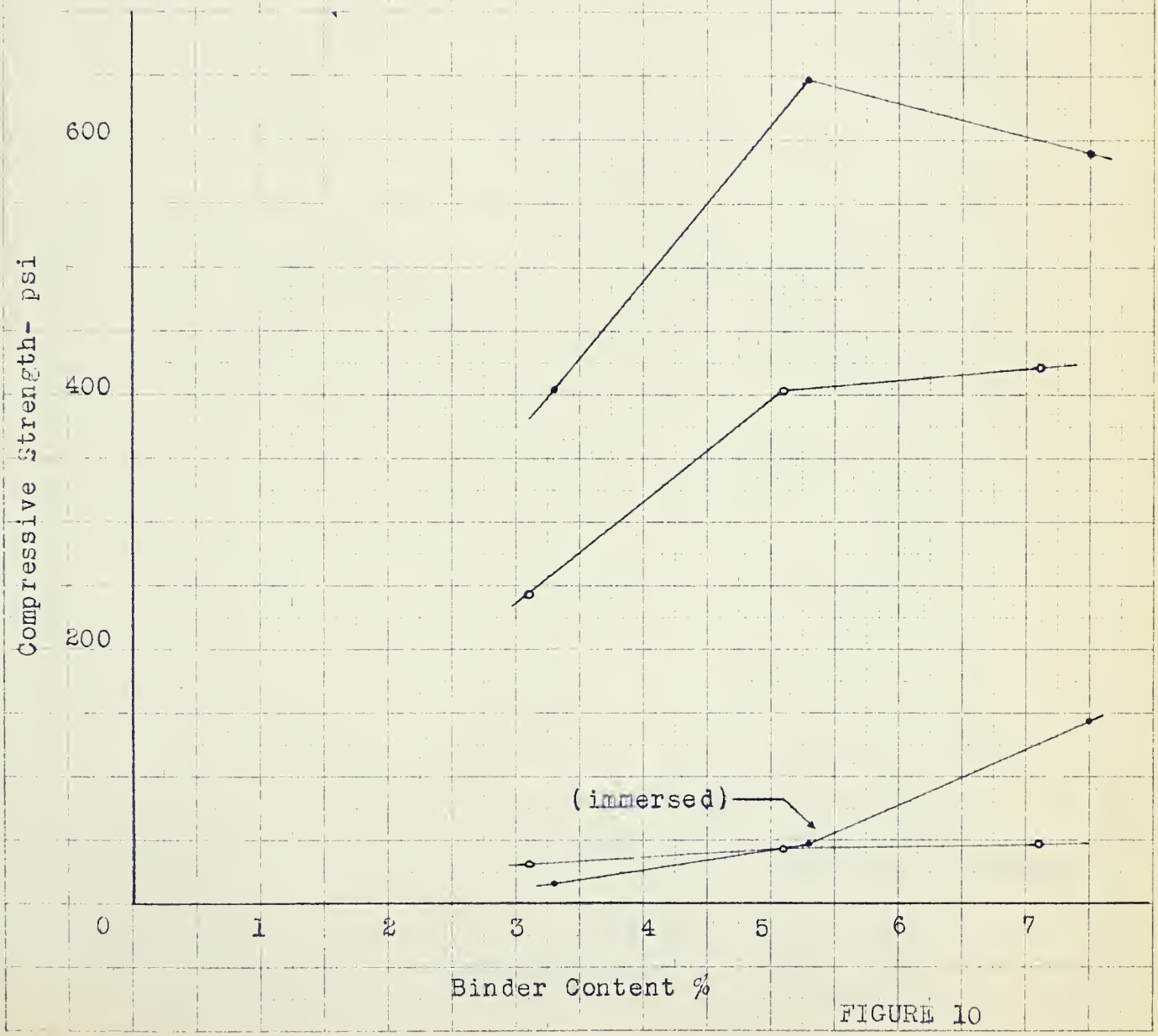

FIGURE 10

BINDER CONTENT VS. COMPRESSIVE STRENGTH

OF COLD FOAM MIXES

CFA-- ×
CFB-- ⊗

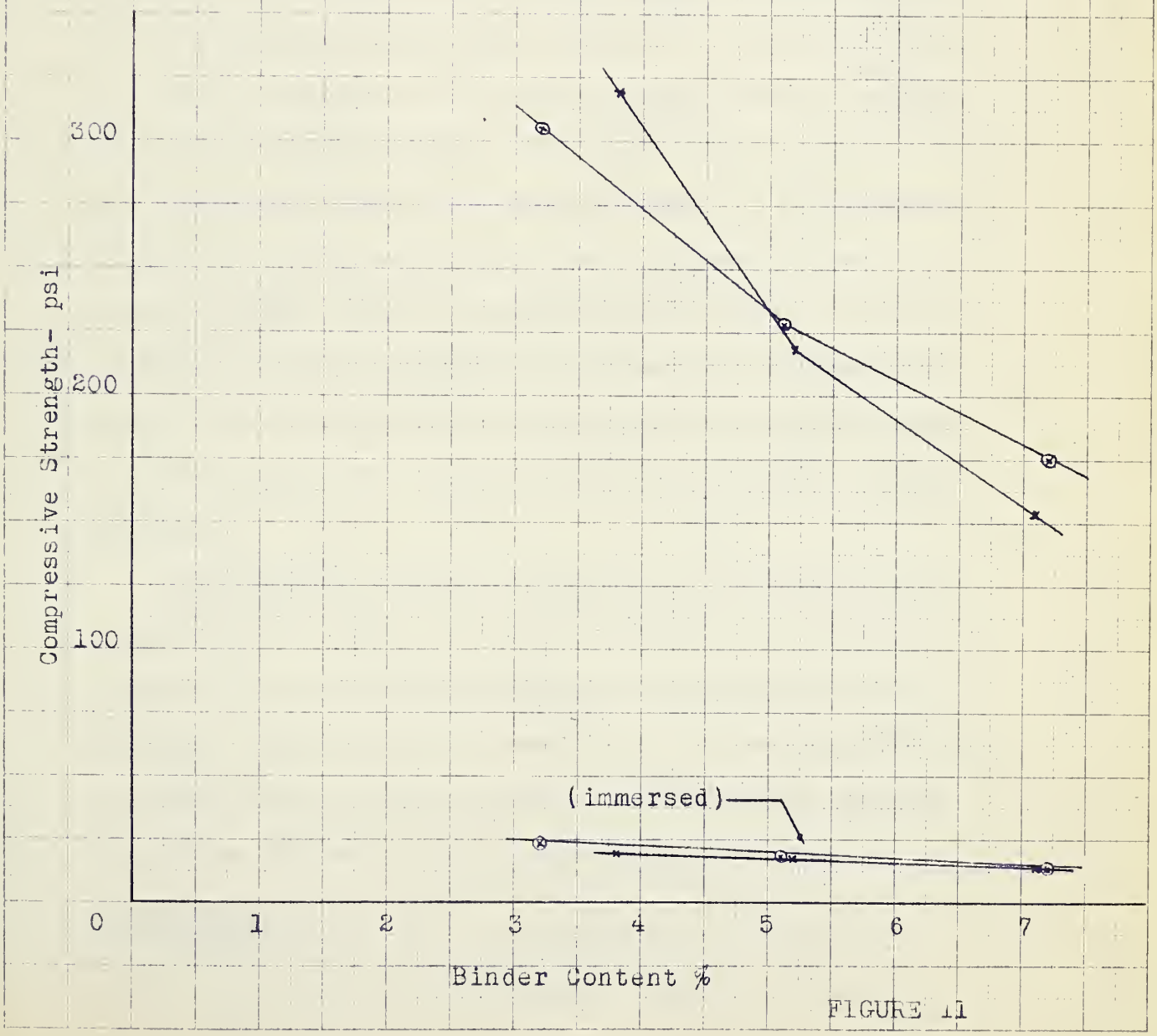

FIGURE 11

two gradations were approximately the same at all binder contents. Dry strength varied from about 300 psi at three percent to 150 psi at seven percent binder content and soaked strengths from 25 psi to 15 psi.

As mentioned previously, the VMA of the cold foam mixes was increasing with increasing binder content[a]. Increased VMA means decreased friction and interlocking between particles and hence decreased frictional strength. This may have been partly responsible for minimum dry strength at the highest binder content and maximum dry strength at the lowest binder content.

The dry strengths of the cold foam mixes were only slightly below those of the hot mix and hot foam mix but the soaked strengths were considerably lower. One possible reason that the dry strength of the cold foam mix was not higher was the presence of about 1.5 percent moisture present in the sample after curing. This may be sufficient moisture to prevent the specimen from exhibiting any apparent cohesion. The moisture could also have remained on the aggregate and prevented a firm bond between the asphalt and the aggregate.

The percent of strength retained by a sample when subjected to the action of water shown in Table IV gives an indication of the damage caused to the mix by stripping of the asphalt from the aggregate. From this point of view HB7 was the best mix, retaining 30 percent of its original strength, HFA7 next, with 24 percent, with CA3 and CB3 poorest, disintegrating when immersed.

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a See page 46

However, the absolute values must also be considered in order to obtain the complete picture. Although HB7 retained 30 percent of its dry strength, the soaked strength was only 89 psi. On the other hand HFA7 retained only 24 percent of its dry strength, but had a soaked strength of 144 psi, considerably higher than the hot mix.

Granulometry. Figures 12 and 13 were plotted in order to determine whether a straight line relationship between compressive strength and cement/void ratio existed as suggested by Winterkorn[a] (6). It can be seen from these figures that as the cement/void ratio increased, the strengths of the hot, hot foam and cold mixes increased to a maximum value at a cement/void ratio of about 0.5, then subsequently decreased. The cold foam mixes, compacted near saturation, showed quite different trends; the strengths decreased steadily as the cement/void ratio increased.

Agreement, within experimental error, was exhibited by similar gradations of the hot and hot foam mixes. Generally, for a given cement/void ratio, higher strengths were produced by gradation A. The cold mixes showed considerably lower strengths but a general tendency to parallel the trends of the hot and hot foam mixes.

Lower strengths in the cold mixes can be explained by the fact that cutback asphalt is of lower viscosity, hence a weaker cementing agent than asphalt cement.

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a See Page 7

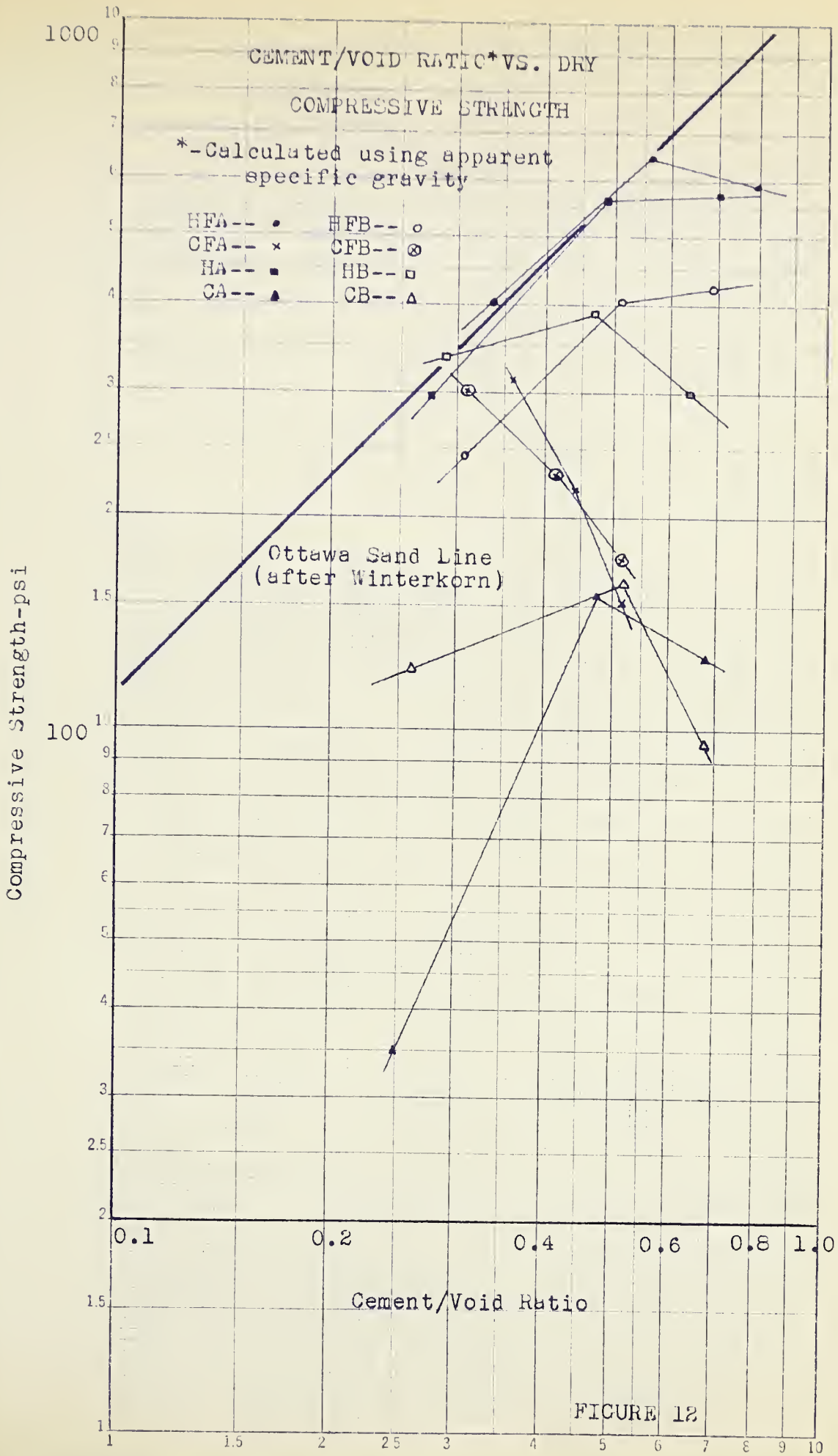
CEMENT/VOID RATIO*VS. DRY
COMPRESSIVE STRENGTH
*-Calculated using apparent specific gravity
HFA-- •
CFA-- ×
HA-- ■
CA-- ▲
HFB-- ○
CFB-- ⊗
HB-- □
CB-- △
Ottawa Sand Line
(after Winterkorn)
Compressive Strength-psi
1000
100
0.1
0.2
0.4
0.6
0.8
1.0
Cement/Void Ratio

FIGURE 12

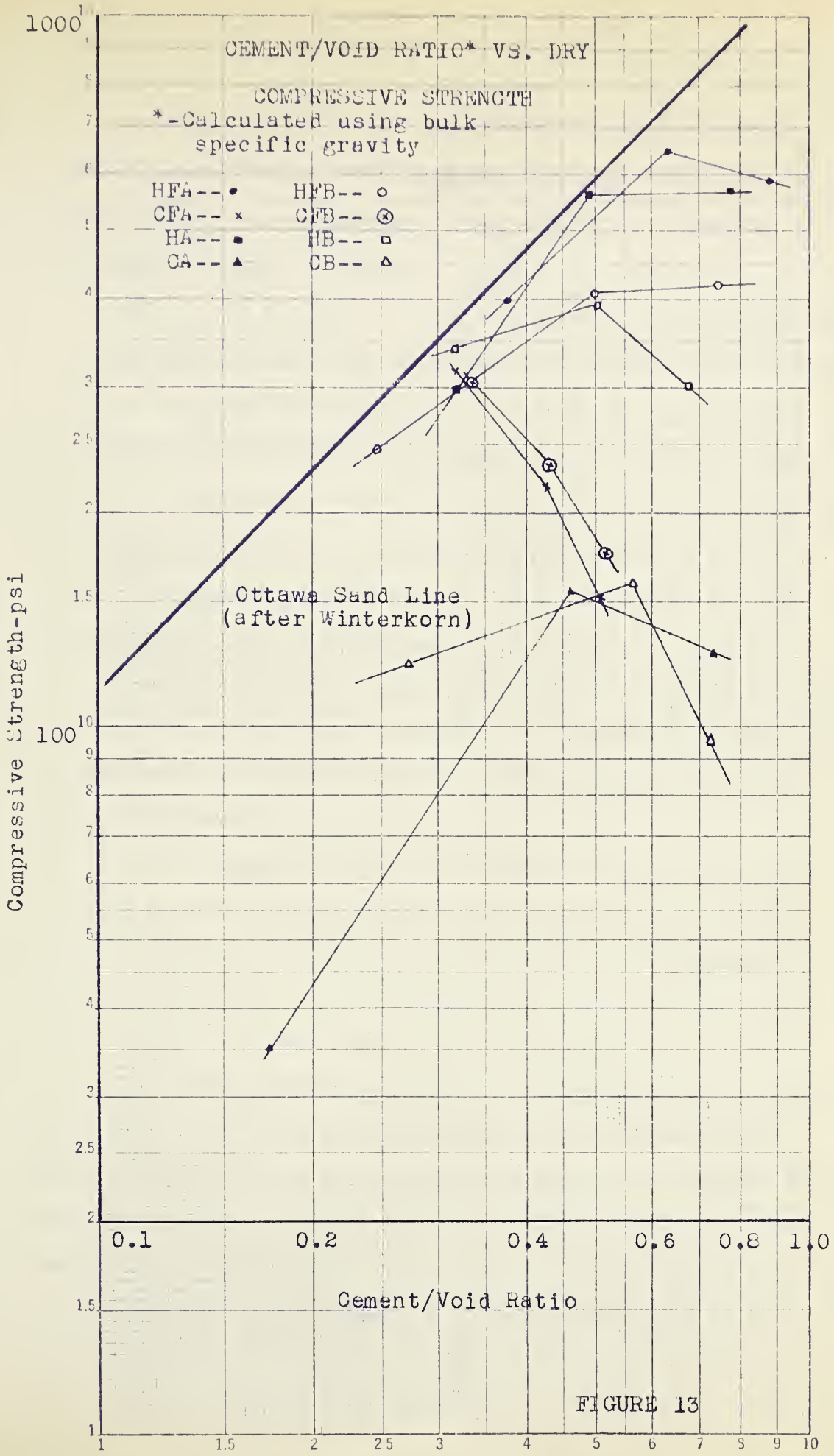
CEMENT/VOID RATIO* VS. DRY
COMPRESSIVE STRENGTH
*-Calculated using bulk specific gravity
HFA-- •
HFB-- ○
CFA-- ×
CFB-- ⊗
HA-- ■
HB-- □
CA-- ▲
CB-- △
Ottawa Sand Line
(after Winterkorn)
Compressive Strength-psi
1000
100
0.1
0.2
0.4
0.6
0.8
1.0
Cement/Void Ratio
1
1.5
2
2.5
3
4
5
6
7
8
9
10

FIGURE 13

For a general comparison, the plot developed for Ottawa sand cemented by analine and furfural compositions was shown. The substances form a strong cement and cause strength increases over a wide range of binder contents.

This is not so with cementing materials which are essentially high-viscosity liquids, such as bitumens. Winterkorn (6) has stated that for bituminous mixtures, the more densely the aggregate particles are packed the higher the strength of the specimen. Since asphalt acts similarly to water in compaction, it is to be expected that aggregate density and strength will pass through a maximum value with increasing binder content. This may explain why the strengths in Figures 12 and 13 pass through a maximum value.

Examination of Table IV shows that the VMA of gradation A is generally lower than that of gradation B. Hence the aggregate density of gradation A is generally higher. This may explain the higher strengths produced by HFA and HA.

When the soaked strengths were plotted in Figures 14 and 15, a linear relationship through all binder contents was obtained. Again, the hot and hot foam mixes showed fairly close agreement, the cold mixes showed similar trends, but at lower strengths and the cold foam mixes showed quite different trends.

For the hot, hot foam and cold mixes, increased binder content has resulted in increasing water-tightness and hence increased soaked strength. It is not known whether the soaked strengths would achieve a maximum value with increased asphalt content in the same manner as the dry strength.

Factors affecting design. The molding method used is said

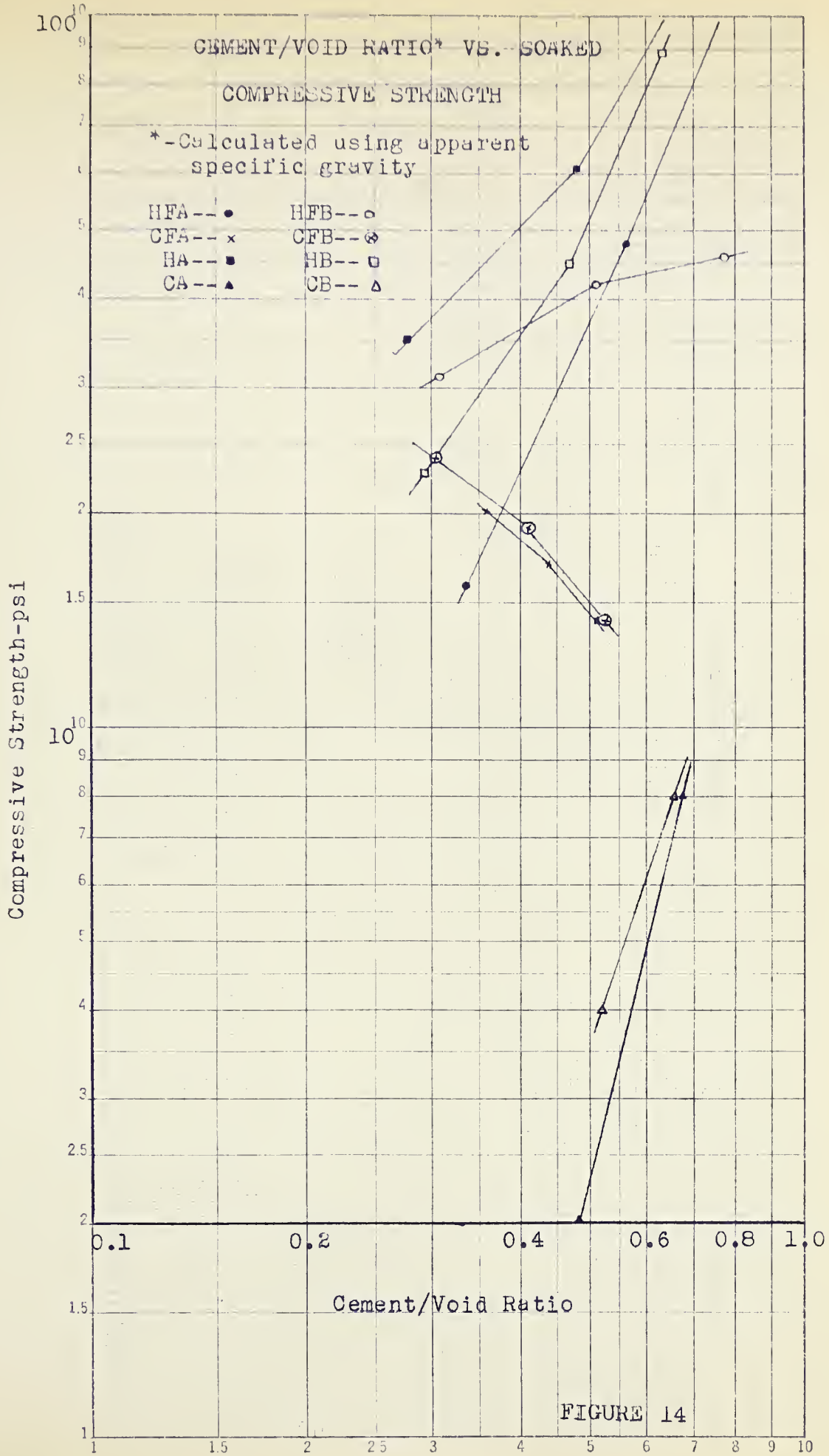
CEMENT/VOID RATIO* VS. SOAKED
COMPRESSIVE STRENGTH
*-Calculated using apparent specific gravity
HFA--●
HFB--○
CFA--×
CFB--⊗
HA--■
HB--□
CA--▲
CB--△
Compressive Strength-psi
Cement/Void Ratio
0.1
0.2
0.4
0.6
0.8
1.0

FIGURE 14

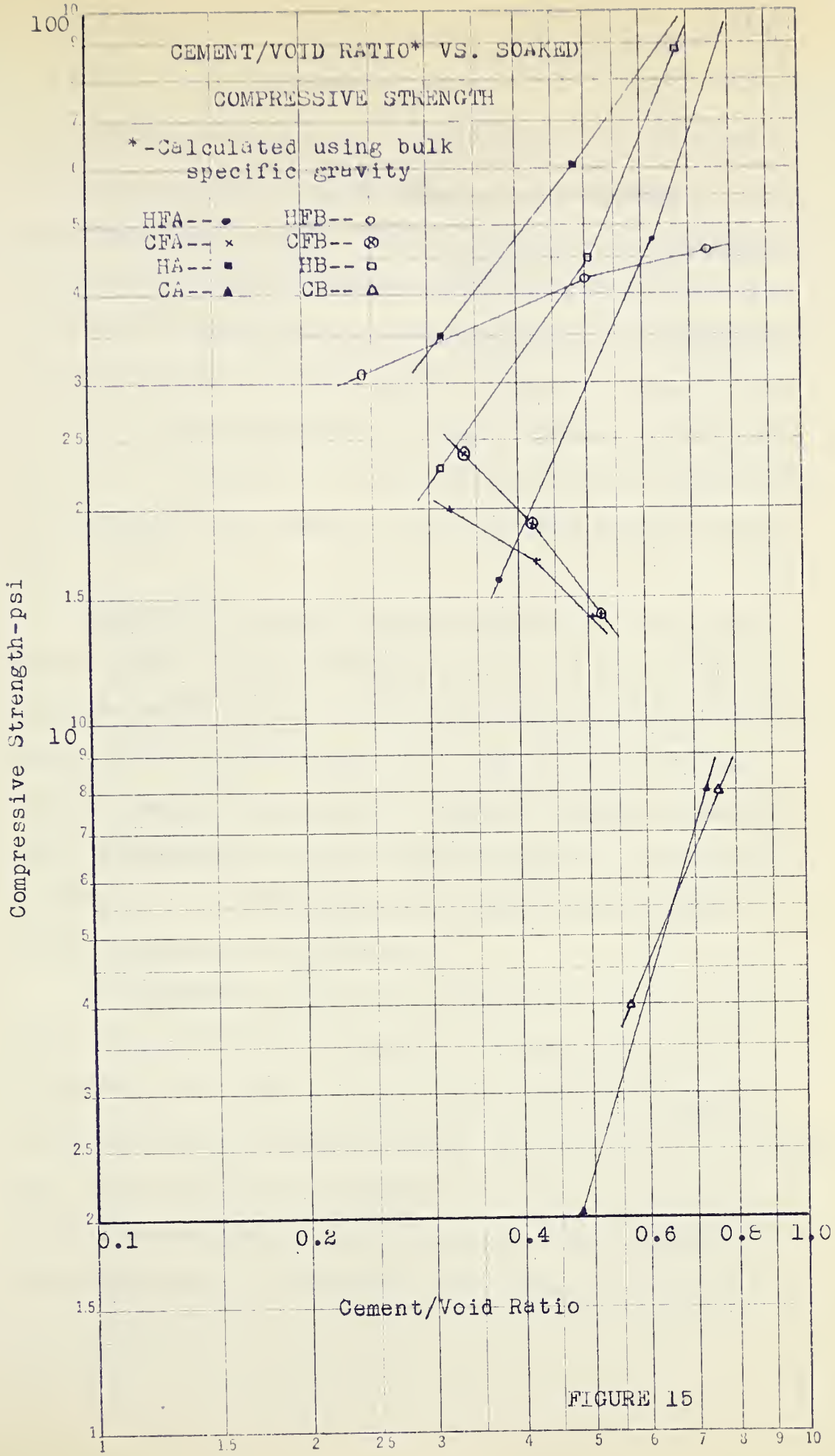
CEMENT/VOID RATIO* VS. SOAKED
COMPRESSIVE STRENGTH
*-Calculated using bulk specific gravity
HFA-- ●
HFB-- ○
CFA-- ×
CFB-- ⊗
HA-- ■
HB-- □
CA-- ▲
CB-- △
Compressive Strength-psi
Cement/Void Ratio
0.1
0.2
0.4
0.6
0.8
1.0

FIGURE 15

to have been sufficiently correlated with field performance to show that it produces laboratory densities corresponding to field densities of dense graded surface mixtures after about one year of traffic. These densities would be considerably greater than those produced in an asphalt stabilized base constructed of this material. Since lower densities produce lower strengths, the strength values obtained for these mixes may be too high. Fractures of the coated particles, which produces discontinuities in the binder where stripping may start, would likely be less in the less densely compacted mix.

The immersed specimens contained from two to eight percent moisture after four days immersion. McLeod (35) reports that non-water-proofed base courses by capillary absorption reach an equilibrium moisture content of from four to seven percent. The same material stabilized with one to two percent asphalt contained from one to three percent moisture. Thus, it seems that the immersion conditions may be overly severe to duplicate field conditions, except in extremely moist climates.

It is generally agreed that no simple relationship exists between water absorption and stability, or swell and stability. As a broad rule, the higher the water absorption, the lower the stability, but there are many other factors which complicate the relationship. Swell is important in itself as affecting distortion of the road surface and subsequent riding quality, hence should be considered principally in that light. Swell is generally thought to

be a complex function of the permeability and water affinity of the mix but as such is not widely used as a design criteria.

Minimum strength requirements exist for asphalt concrete surfacing mixtures (22). They are 200 psi dry strength and 70 percent strength retained after immersion (140 psi soaked strength). In light of this, the only satisfactory mixture of the entire series tested was HFA7 with a soaked strength of 144 psi. Strength requirements for asphalt treated base courses would undoubtedly be lower due to lower stresses and temperature extremes (36) present in base courses. A more complete discussion of this subject is contained in reference number 36.

A factor which probably caused lower strength was the presence of dust on the aggregate particles, impairing coverage and adhesion. The capillary passages in the dust would in turn aid the entrance of water and stripping. To this end, design requirements usually limit the dust/asphalt ratio[a] to 1.2 for high standard asphalt mixes (22). The range of dust/asphalt ratios used varied from 7.5 for CA3 to 1.7 for HFB7.

The test results showed the two hot mixes superior to the cold mixes. They are also more expensive to produce, requiring extensive heating of the aggregate. The cold foam mix seemed to be superior to the cold mix. It is felt that higher strengths could have been obtained from the cold foam mix. By decreasing the mixing moisture content, higher densities could be achieved, providing higher frictional strength. Increased strength might also be achieved by in-

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a The percentage of aggregate passing the No.200 sieve divided by the percentage of asphalt.

creasing the curing of the mixture in an attempt to drive off more moisture. Then higher capillary forces would be in effect, giving higher dry strengths.

Summary. The density of the cold foam mixes decreased with increasing binder content while the other mix types increased in density through the range of binder used. Mix types listed in general order of decreasing density produced were cold mix, hot foam, hot and cold foam mixes. It was generally noted that gradation A produced the highest density for a particular mix type.

An increase in binder content reduced the amount of water absorbed by the hot foam and cold mixes by a considerable amount. A similar increase in binder content in the cold foam mixes produced only slight reduction in the amount of absorbed water. In general it was noted that for a given amount of water absorption that the cold mixes swelled the largest amount, the hot foam mixes next, and the cold foam mixes least.

The dry and soaked strengths of the cold foam mixes decreased with increasing binder content. The other mix types generally attained the maximum dry strength at about five percent binder and the maximum soaked strength at about seven percent binder. In general gradation A produced higher strengths. Mix types listed in order of decreasing magnitude of maximum dry strength were hot foam mix, hot, cold foam and cold mixes. When the compressive strength was plotted versus cement/void ratio on a logarithmic plot, the hot, hot foam and cold mixes showed trends similar to those proposed by Winterkorn (6).

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

From the laboratory investigation and the analysis of the data, the following conclusions are reached:

a. All mixes react poorly to the action of water with the percent of strength retained varying from 0 to 30 percent. According to the strength criteria used[a] only one mix, HFA7, has suitable soaked strength for use as a surfacing material. Since stresses are lower in base courses, other mixes with high soaked strengths, such as HB7, would undoubtedly be suitable in base course also.

b. The hot foam mix gives the best performance, with hot mix, cold foam mix and cold mix following in that order.

c. Maximum dry strength generally occurs at a minimum VMA, the condition in which the aggregate particles are most closely packed. This condition generally occurs at a binder content below that necessary for maximum density of the total mix. This indicates that for maximum dry strength, friction is important.

d. Maximum soaked strength generally occurs when the absorbed water is at a minimum. Increasing the binder content from three to seven percent is effective in reducing the percent voids and absorbed water, and in increasing the soaked strength of the hot mix, the hot foam mix, and the cold mix. Because of the excess water in the cold foam mixes, a similar binder content increase results in very little change in percent voids and absorbed water, and causes a decrease in

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a See page 69

soaked strength.

e. Gradation A, the material which falls outside AASHO base course requirements, produces higher densities and higher strengths than gradation B. Thus, gradation A produces higher strengths in the stabilized material at the risk of subsequent damage due to swell.

Recommendations for Future Research

a. The mixtures should be mixed in the same pugmill in order to reduce the number of variables present. The MC3 cutback in the cold mix and the asphalt cement in the hot mix should be introduced into the aggregate in some manner, such as spraying, to better reproduce field mixing.

b. An investigation is required into the effect of mixing moisture content upon the density and strengths of the cold mix and the cold foam mix at various asphalt contents. For this investigation, unconfined compression tests would serve to determine the basic trends present.

c. In order to obtain a better quantitative comparison of the different mix types, triaxial tests are required. Using this test, a check can be run to determine whether the immersion compression test actually does provide a reliable measure of the cohesion lost due to the action of water.

d. Further tests are required to investigate the relationship proposed by Winterkorn[a]. Tests over a broader range of various asphalt contents should be run to determine the relationship more

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a See page 7

closely. A series of tests using various types of soil and asphalt is required to determine whether the relationship proposed by Winterkorn does exist.

e. In order to correlate test results with actual performance, a study of the use and performance of this aggregate in bituminous mixtures in Alberta should be made. Test results could then be compared with actual behaviour and test procedure modified to give results which agree with observed field behaviour.

f. An investigation is required to determine whether the swelling characteristics exhibited by these mixtures would cause serious damage to a pavement built upon it. Further tests are also required to determine the reaction of the various mixtures to other variations in climatic conditions, such as freezing and thawing.

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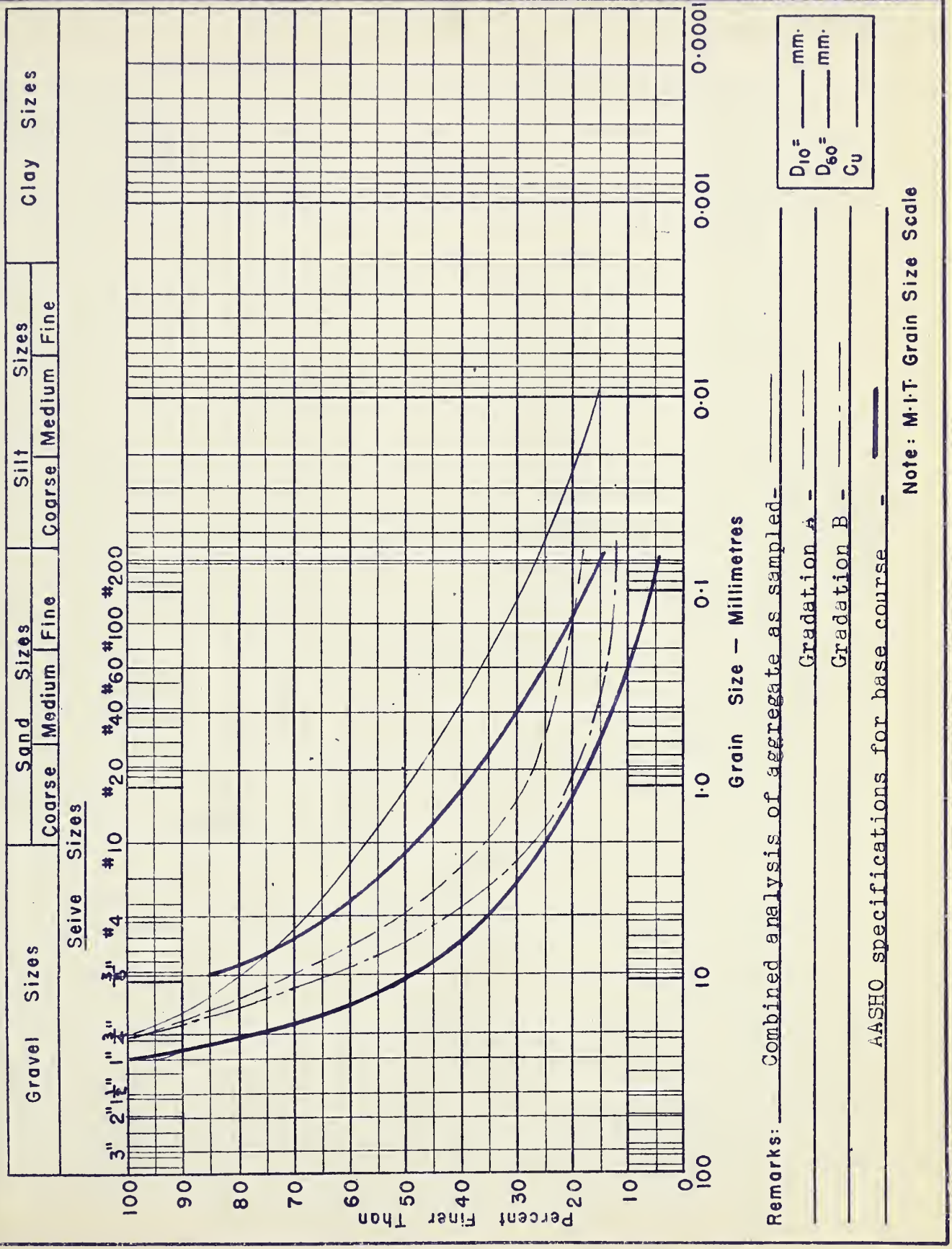
APPENDIX A

RESULTS OF CLASSIFICATION TESTS

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62



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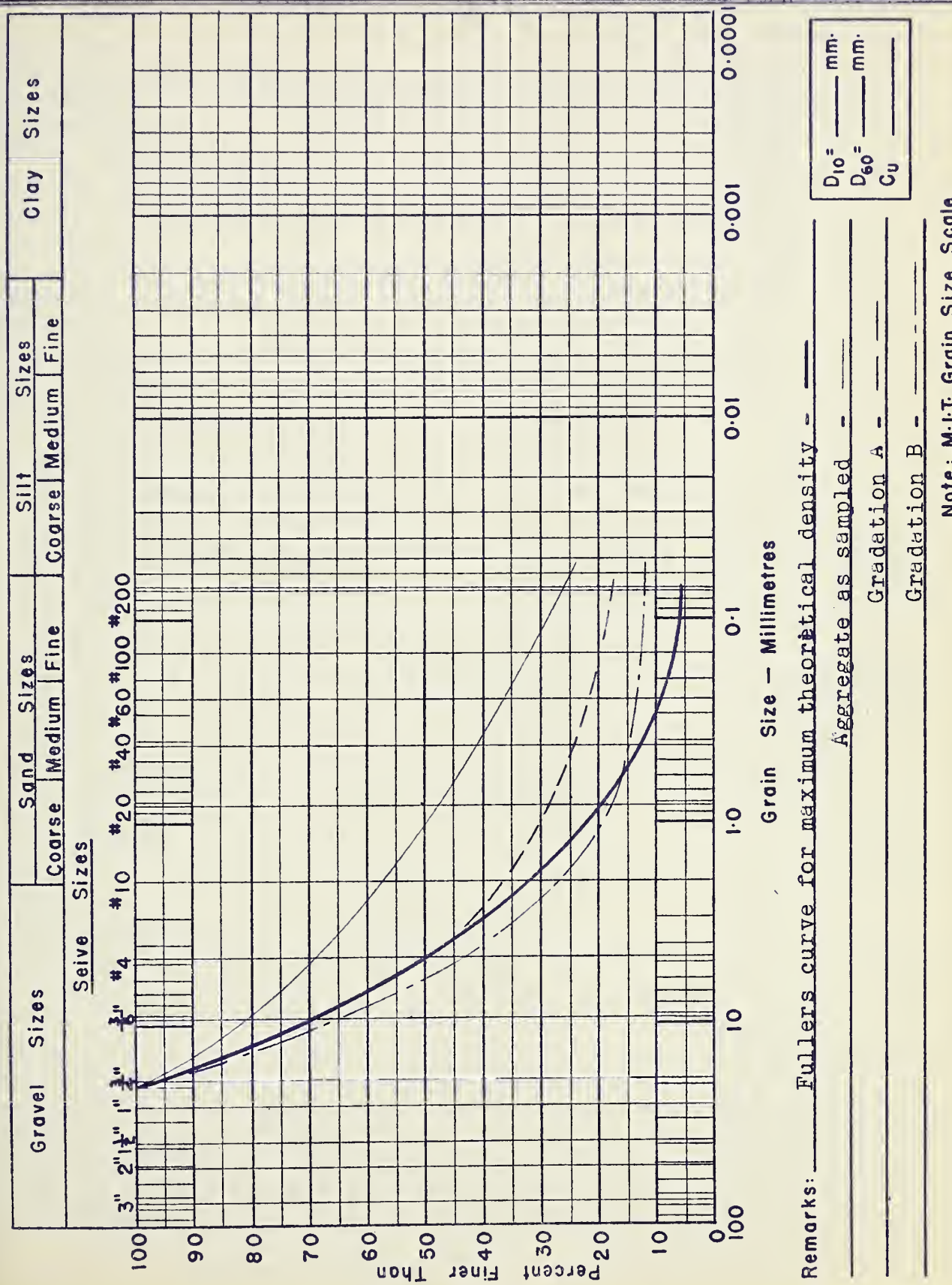
GRAIN SIZE CURVE

PROJECT	Thesis
SITE	Christianson Pit
SAMPLE	Gravel
LOCATION	Carstairs, Alberta
HOLE	DEPTH
TECHNICIAN A.B.	DATE June/62



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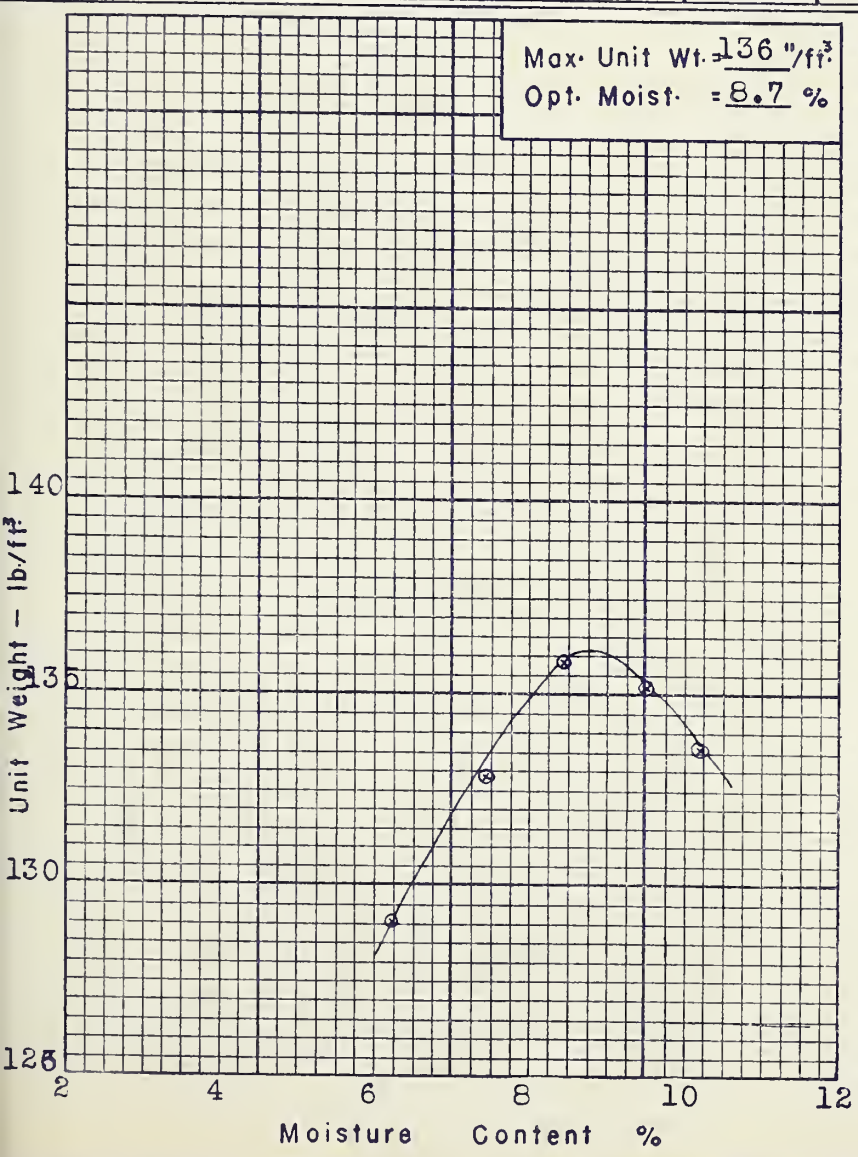
COMPACTION TEST

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION
HOLE A.B. DEPTH June/62
TECHNICIAN DATE

	Trial Number	1	2	3	4	5		
Unit Weight Determination	Mold No.	1	2	3	4	5		
	Wt. Sample Wet + Mold	11,622	11,665	11,601	11,690	11,682		
	Wt. Mold	9,436	9,436	9,436	9,447	9,447		
	Wt. Sample Wet	2,186	2,229	2,165	2,243	2,235		
	Volume Mold	1/30	1/30	1/30	1/30	1/30		
	Wet Unit Weight lb./ft.^{3}	144.3	147.1	142.9	148.0	147.2		
	Dry Unit Weight lb./ft.^{3}	129.0	135.9	132.9	135.1	133.6		
Moisture Content Determination	Container No.	1	2	3	4	5		
	Wt. Sample Wet + Tare	1041.3	1777.9	1141.6	2173.8	1834.4		
	Wt. Sample Dry + Tare	987.4	1650.3	1072.1	1997.2	1683.1		
	Wt. Water	53.9	127.6	69.5	176.6	151.3		
	Tare Container	114.9	138.1	133.2	44.0	179.6		
	Wt. Dry Soil	872.5	1512.2	938.9	1853.2	1503.5		
	Moisture Content	6.2	8.4	7.4	9.5	10.2		



Method of Compaction
~~Standard Proctor~~

Diam. Mold 4.0 inches
Height Mold 4.6 inches
Volume Mold 1/30 ft.^{3}
No. of Layers 3
Blows per Layer 25
Ht. of Free Fall 12 inches
Wt. of Tamper 5.5 lbs
Shape of Tamping Face 0
Description of Sample
Gradation A

Remarks

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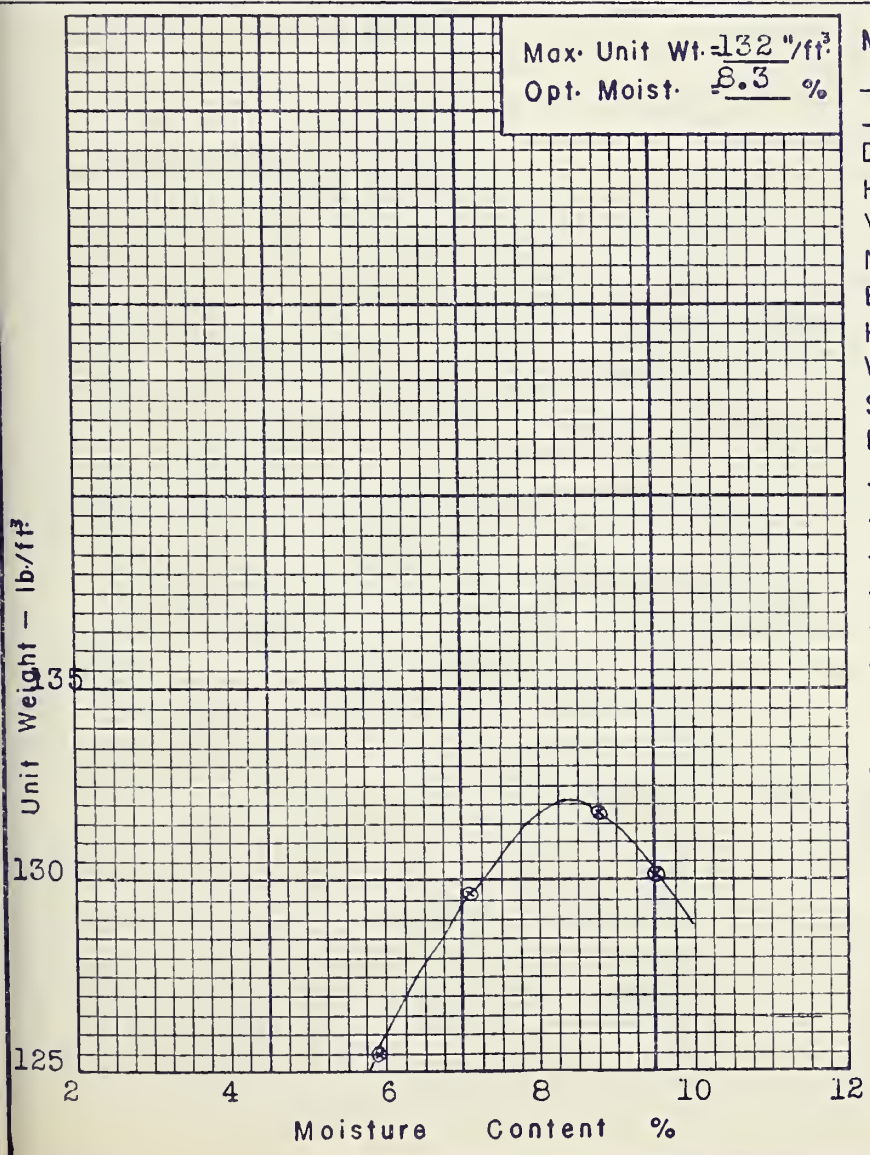
COMPACTION TEST

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

	Trial Number	1	2	3	4			
Unit Weight Determination	Mold No.	1	2	3	4			
	Wt. Sample Wet + Mold	11,485	11,590	11647	11,636			
	Wt. Mold	9,475	9,475	9,475	9,475			
	Wt. Sample Wet	2,010	2,115	2,172	2,161			
	Volume Mold	1/30	1/30	1/30	1/30			
	Wet Unit Weight lb./ft.^{3}	132.5	139.6	143.3	142.8			
	Dry Unit Weight lb./ft.^{3}	125.5	130.1	131.9	130.1			
Moisture Content Determination	Container No.	1	2	3	4			
	Wt. Sample Wet + Tare	2041.2	2131.0	2196.8	2156.9			
	Wt. Sample Dry + Tare	1940.0	1998.0	2032.5	1979.7			
	Wt. Water	101.2	133.0	164.3	177.2			
	Tare Container	120.0	133.2	138.1	114.9			
	Wt. Dry Soil	1820.0	1864.8	1894.4	1864.8			
	Moisture Content	5.9	7.1	8.7	9.5			



Method of Compaction
Standard Proctor

Diam. Mold 4.0 inches
Height Mold 4.6 inches
Volume Mold 1/30 ft.^{3}
No. of Layers 3
Blows per Layer 25
Ht. of Free Fall 12 inches
Wt. of Tamper 5.5 lbs
Shape of Tamping Face o
Description of Sample

Gradation B

Remarks

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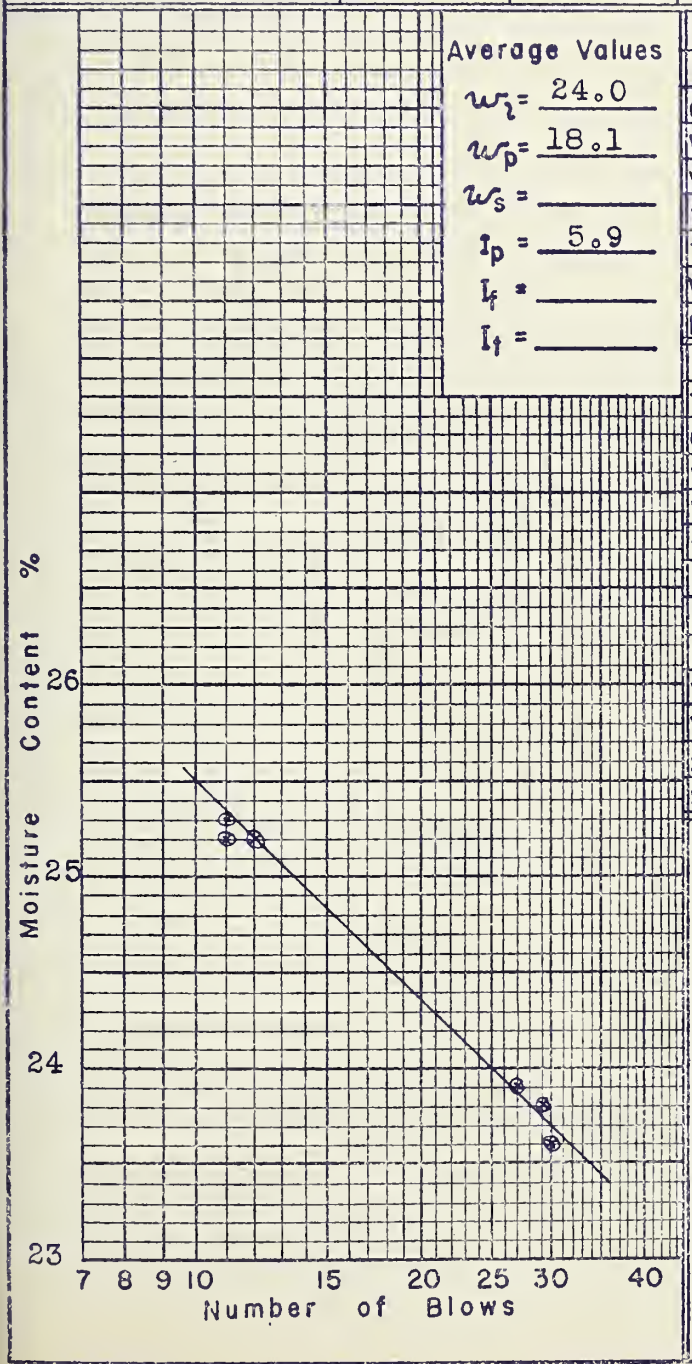
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

Liquid Limit

Trial No.	1	2	3	4	5	6
No. of Blows	11	11	12	27	29	30
Container No.	1	2	3	4	5	6
Wt. Sample Wet + Tare	85.13	96.22	88.55	94.93	81.64	99.79
Wt. Sample Dry + Tare	83.09	93.84	86.31	92.65	79.83	96.90
Wt. Water	2.04	2.38	2.24	2.28	1.81	2.89
Tare Container	75.04	84.42	77.41	83.13	72.24	84.66
Wt. of Dry Soil	8.05	9.42	8.90	9.52	7.59	12.24
Moisture Content w%	25.2	25.3	25.2	23.9	23.8	23.6



Average Values
w_l = 24.0
w_p = 18.1
w_s = ____
I_p = 5.9
I_f = ____
I_t = ____

Plastic Limit

Trial No.	7	8	9
Container No.	7	8	9
Wt. Sample Wet+Tare	98.50	81.22	70.51
Wt. Sample Dry+Tare	97.43	78.05	69.34
Wt. Water	1.07	3.17	1.17
Tare Container	91.60	60.50	62.76
Wt. of Dry Soil	5.83	17.55	6.58
Moisture Content %	18.4	18.1	17.8

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: Air-dried material passing the No. 40 sieve

Remarks:

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SPECIFIC GRAVITY

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

Sample No.	1	2
Flask No.	1	2
Method of Air Removal	Vacuum	Vacuum
W_{b+w+s}	765.69	725.48
Temperature T	24.1	24.3
W_{b+w}	680.27	655.43
Evaporating Dish No.	3	4
Wt. Sample Dry + Dish	505.62	481.07
Tare Dish	370.42	369.89
W_s	135.20	111.18
G_s	2.72	2.71

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles = $\frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample: Retained on No. 10 sieve

Remarks: Average value of 3 determinations = 2.72

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SPECIFIC GRAVITY

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

Sample No.	1	2
Flask No.	1	2
Method of Air Removal	Vacuum	Vacuum
W_{b+w+s}	714.11	706.60
Temperature T	24.9	23.9
W_{b+w}	680.18	655.48
Evaporating Dish No.	4	5
Wt. Sample Dry + Dish	424.29	457.24
Tare Dish	370.58	376.49
W_s	53.71	80.75
G_s	2.72	2.73

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles = $\frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample: Pass No. 10 sieve

Remarks: Average value of 3 determinations = 2.72

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SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

Date	Temp.	Time	Elapsed Time	R_h'	$R_h = R_h' + c_m$	D m.m.	$R_h + m_t - c_d$	W %	W % Basis Orig. Sample	Remarks
	°C		min.							
9 June	22.5	0951	0	-	-	-	-	-		
	22.5		1/4	27.0	27.3	.090	28.1	71.2		
	22.5		1/2	25.5	25.8	.064	26.6	67.4		
	22.5	0952	1	23.3	23.6	.048	24.4	61.7		
	22.5	0953	2	21.1	21.4	.036	22.2	56.2		
	22.7	0955	4	19.8	20.1	.024	21.0	53.2		
	22.8	0959	8	18.0	18.3	.017	19.2	48.6		
	23.0	1007	16	16.3	16.6	.0125	17.5	44.4		
	23.0	1023	32	14.4	14.7	.0092	15.6	39.5		
	23.0	1151	60	12.9	13.2	.0068	14.1	36.7		
	23.0	1226	155	10.6	10.9	.0043	11.8	30.2		
	22.7	1600	369	8.9	9.2	.0028	10.0	25.3		
	22.8	2327	816	7.2	7.5	.0020	8.4	21.2		
10 June	22.3	0930	1419	6.2	6.5	.0015	7.2	18.2		
11 June	22.3	0910	2839	5.4	5.7	.0011	6.4	16.2		
13 June	22.9	1735	6874	4.9	5.2	.00005	6.1	15.4		

Hydrometer No.s 36067 and 21280 Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1} (R_h + m_t - c_d) = 2.51 \ (R_h + m_t - c_d)$

Meniscus correction = c_m = 0.3 and 0.4 respectively

Dispersing agent used Calgon Amount 10 cc

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.4 and 0.1 respectively

Specific Gravity of Solids = G_s 2.72

Description of Sample
Pass No. 40 sieve

Method of Preparation
100 cc of Hydrogen Peroxide and 10 cc of Calgon added
Agitated for 5 minutes

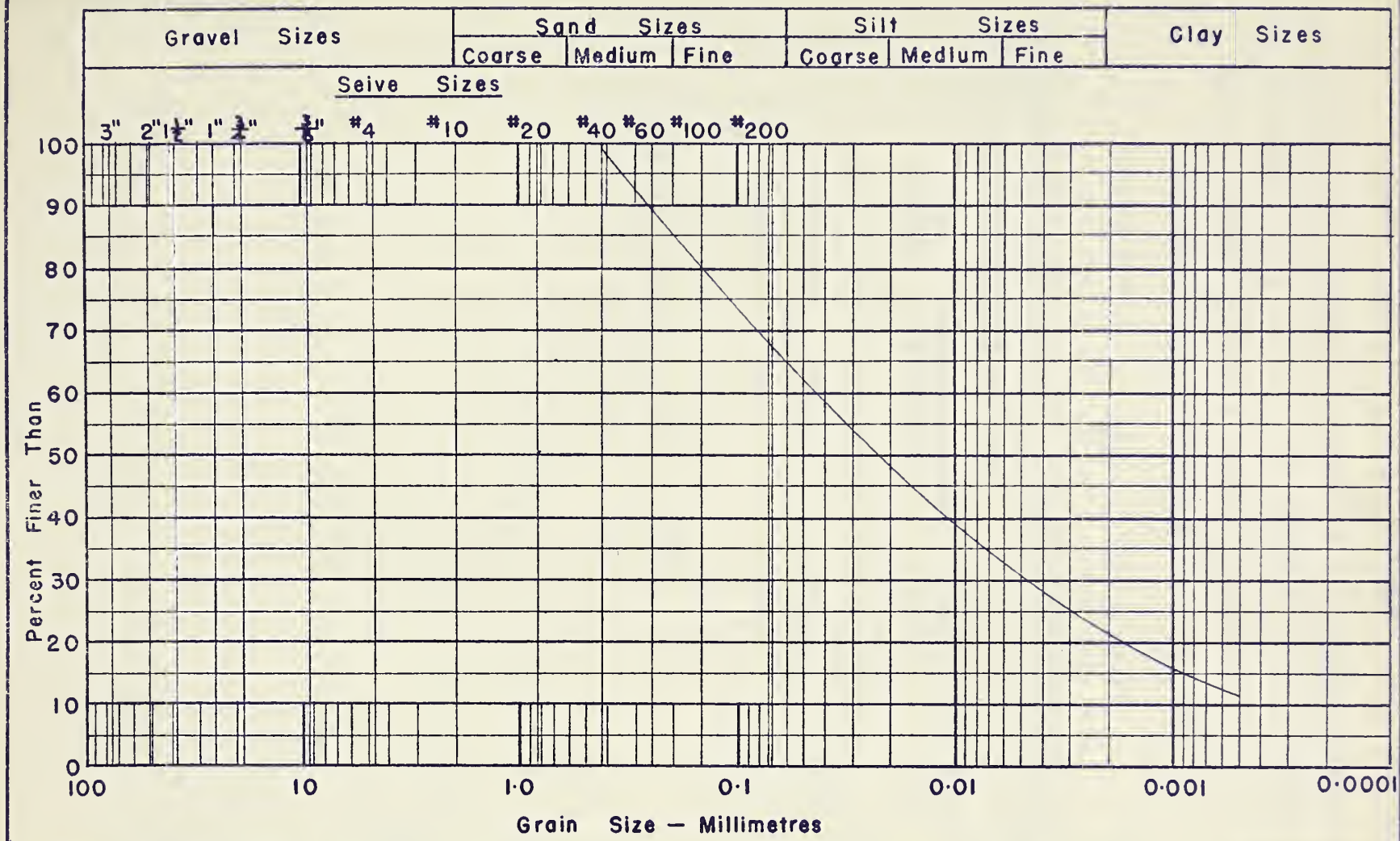
Remarks
Trial 1 of 3 trials

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No. 1
Wt. Sample (Wet/Dry) + Tare 141.35
Tare 78.26
Wt. (~~Wet~~/Dry) Soil 63.09
Dry Weight from Initial
Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist. \%}}$ =



Remarks: Average results of 3 hydrometer analyses on material passing the No. 40 sieve

D_{10} = ____ mm.
D_{60} = ____ mm.
C_u ____

Note: M·I·T· Grain Size Scale

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GRAIN SIZE CURVE

PROJECT Thesis
SITE Christianson Pit
SAMPLE Gravel
LOCATION Carstairs, Alberta
HOLE DEPTH
TECHNICIAN A.B. DATE June/62

Los Angeles Abrasion Test

Weight of aggregate used was 5000 gms. composed of 2500 gms passing the $\frac{3}{4}$ inch sieve and retained on the $\frac{1}{2}$ inch sieve and 2500 gms passing the $\frac{1}{2}$ inch sieve and retained by the 3/8 inch sieve. A charge of 11 balls was used with 500 revolutions.

Weight of aggregate lost due to testing = 1227 gms.

Percent loss = $\frac{1227}{5000} \times 100 = 24.5\%$

Determination of Bulk Specific Gravity and Water Absorption of Material Retained on No.10 Sieve

Sample No.	Dry Weight	Saturated Surface Dry Weight	Immersed Weight	Bulk Specific Gravity	Percent Water Absorbed
1	596.2	607.0	375.1	2.57	1.8
2	602.0	614.6	378.8	2.56	2.1
3	535.4	547.3	336.5	2.55	2.0

Average bulk specific gravity = 2.56

Average percent water absorbed = 2.0%

APPENDIX B

DESCRIPTION OF THE PUGMILLS USED

DESCRIPTION OF PUGMILLS

The University pugmill has a ½ hp motor with a direct gear connection to the shafts. It is a twin shaft type with six paddles set into each shaft. Each paddle is staggered about 60°, giving a "corkscrew" effect which tends to move the soil to the front of the machine. Mixing speed cannot be varied and is about 95 revolutions per minute. Paddles are 2½ inches long by 2 inches wide with a 3½ inch radius measured from the centre of the shaft. Overall dimensions of the mixing chamber is 15 inches long by 11 inches wide by 13 inches high. There are electric heaters on each end capable of heating the unit to about 250°F.

The B.C.H. pugmill is driven by a ½ horsepower motor connected by a variable speed belt drive. Normal mixing speed is about 140 revolutions per minute. There are seven paddles, each staggered 180°, on each of the two shafts. The paddles are 2½ inches long by 2 inches wide on a 3½ inch radius measured from the centre of the shaft. Overall dimensions of the mixing chamber are 17 inches long by 15 inches wide by 16 inches high. The foam nozzle is located about 14 inches above the shafts. Steam is supplied from a gas heated boiler and the asphalt from an electrically heated container. Steam pressure is 50 psi and asphalt pressure 28 psi. The asphalt is metered volumetrically by means of a float with a pointer to indicate its position. Asphalt pressure is supplied by a small pump unit run by a separate motor. Electric heaters on each end are capable of heating the unit to 280°F.

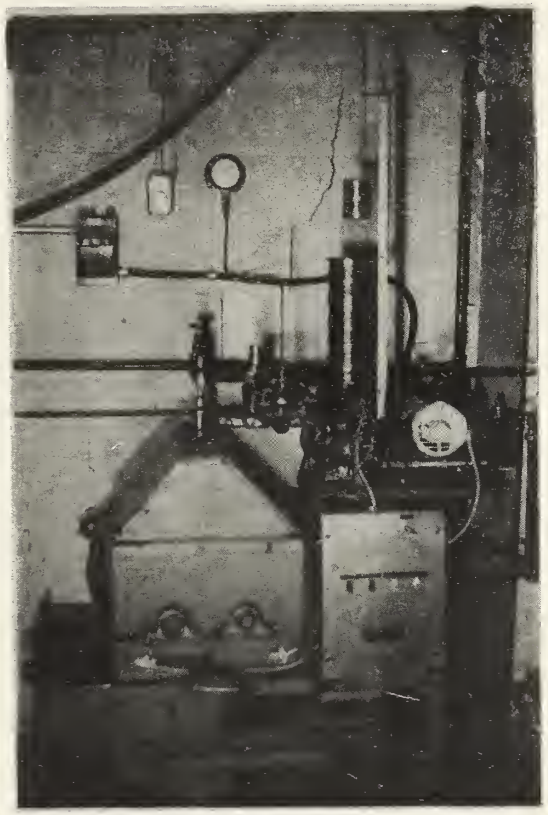

Photo 4. B. C. H. Pugmill

Photo 5. University Pugmill

APPENDIX C

PROCEDURE USED FOR PETROGRAPHIC ANALYSIS

Procedure for Petrographic Analysis

The equipment necessary consists of a pen knife, dilute hydrochloric acid, a lens, a hammer and a splitting block. The gravel sample of 1000 gms was made up of retained $\frac{1}{2}$ inch and 3/8 inch aggregate proportioned in the same ratio as the combined weight of the $\frac{3}{4}$ inch plus $\frac{1}{2}$ inch material bears to the 3/8 inch material. The analysis of sand was performed on 500 gms of the retained No.4 material if the retained No.4 material exceeds 10% of the total sample. The samples were washed (for easier identification) except when the sample contained a high percentage of clay or clay balls.

In the case of sedimentary aggregates, the dolomites and limestones were separated into angular and pebble portions. A further separation was then made on the basis of soundness, weathering and hardness. The presence of clay, shale or chert or the effect of ferrous compounds with clay, as shown by ochrous coloring were then separated. Sandstone, chert or shale were then examined and separated as to composition, hardness and weathering.

A similar procedure was followed in igneous and metamorphic aggregates. The most numerous type was separated first, followed by the less numerous ones. Further separation was made on the basis of structure, grain size, degree of metamorphism, hardness, weathering and decomposition.

When the separation was completed, the fractions were weighed and recorded and the calculations performed. This resulted in the sample being assigned a petrographic number, a numerical quality

appraisal for that particular aggregate.

Petrographic limits for coarse aggregates, as specified by the Ontario Department of Highways are tabulated below.

Type of Construction	Aggregates	Maximum Petrographic Number
Surface Course	Trap rock	100
Surface Course	Commercial	135
Base and surface	Local gravel or rock	160
Concrete Structures and Pavements		
Concrete Culverts		155
Mulch pavement	5/8" crushed	175
Shouldering	5/8" crushed	185

APPENDIX D

SAMPLE CALCULATION

Sample Calculations of Sample Properties

For an example, sample 1 of series BH7 will be selected. For illustration, calculations will be performed using:

(a) the apparent specific gravity,

(b) the bulk specific gravity with allowance for asphalt absorption.

Dry weight of sample after curing = 1813.8 gms

Surface dry weight of specimen = 1823.3 gms

Immersed weight of specimen = 1016.7 gms

Bulk Volume of specimen = 1823.3 - 1016.7 = 806.6 cc.

Asphalt content = 7.0%

Asphalt specific gravity = 1.029

Apparent specific gravity of aggregate = 2.72

Bulk specific gravity of aggregate = 2.56

Asphalt absorption = 1.2%

Weight of soil in sample $= \frac{1813.8}{1.07} = 1698$ gms

Volume of soil in specimen:

(a) $= \frac{1698}{2.72} = 624$ cc.

(b) $= \frac{1698}{2.56} = 663$ cc.

Weight of effective asphalt

(a) = 1813.8 - 1698 = 115.8 gms.

(b) $= \frac{7.0 - 1.2}{7.0} \times 115.8 = 96.0$ gms

Volume of effective asphalt:

(a) $= \frac{115.8}{1.029} = 112.5$ cc

(b) $= \frac{96.0}{1.029} = 93.5$ cc

Volume of air voids:

(a) = 806.6 - (624 + 112.5) = 70.1 cc

(b) = 806.6 - (663 + 93.5) = 50.1 cc

Percent air voids:

$$(a) = \frac{70.1}{806.6} \times 100 = 8.7\%$$

$$(b) = \frac{50.1}{806.6} \times 100 = 6.2\%$$

Percent of volume occupied by the aggregate:

$$(a) = \frac{624}{806.6} \times 100 = 77.4\%$$

$$(b) = \frac{663}{806.6} \times 100 = 82.2\%$$

Percent voids in the mineral aggregate:

$$(a) = 100.0 - 77.4 = 22.6\%$$

$$(b) = 100.0 - 82.2 = 17.8\%$$

Percent of the voids occupied by asphalt:

$$(a) = \frac{112.5}{112.5 + 70.1} \times 100 = 61.7\%$$

$$(b) = \frac{93.5}{93.5 + 50.1} \times 100 = 65.1\%$$

Weight of soaked specimen in air = 1896.3 gms

Weight of water absorbed = 1896.3 - 1823.3 = 73.0 gms

Percent of water absorbed $= \frac{73.0}{1698} \times 100 = 4.3\%$

BITUMINOUS MIX DESIGN

AGGREGATE BLEND

Sieve Size	Aggregate A Pass $\frac{3}{4}$", Retained on No. 4			Aggregate B Passing No. 40, Retained on No. 10			Aggregate C Passing No. 10, Retained on No. 40			Aggregate D Passing No. 200		Desired Gradations		Allowable Gradations Spec. A	Spec. B
	Sieve Analysis	55% A	60% A	Sieve Analysis	20% B	25% B	Sieve Analysis	15% C	15% C	Sieve Analysis	10% D	A	B		
$\frac{3}{4}$"	100	55	60	100	20	25	100	15	15	100	10	100	100	100	100
No. 4	9.8	5.4	5.9	100	20	25	100	15	15	100	10	50	50	50-70	45-70
No. 10	7.8	4.3	4.7	27.4	5.5	6.8	100	15	15	100	10	35	35	30-50	27-50
No. 40	7.1	3.8	4.3	20.0	4.2	5.0	52.7	8.1	8.1	100	10	25	20	20-40	17-40
No. 200*	5.4	2.9	3.2	14.1	2.8	3.5	35.9	5.4	5.4	68	6.8	18	10	15-25	10-25

* Percent passing wet sieve analysis.

APPENDIX E

TEST RESULTS

Data and Results of Immersion-Compression Test

Test CA3 Asphalt Content 2.4

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1758.0	1730.2	1756.4	1742.2	1744.1	1739.7
3. Surface Dry Weight - gm.	1778.8	1758.8	1811.0	1772.8	1786.1	1789.5
4. Immersed Weight - gm.	985.8	986.2	1031.5	994.7	1007.1	1009.9
5. Bulk Volume - c.c.	793.0	772.6	779.5	778.1	779.0	779.6
6. Bulk Specific Gravity - gm/c.c.	2.22	2.24	2.25	2.24	2.24	2.23

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	270	0*	0*	0*	170	160
15. Dry Compressive Strength - psi	47				30	28
16. Soaked Compressive Strength - psi		0	0	0		

*- sample disintegrated

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.24
18. Dry Strength - psi	35
19. Soaked Strength - psi	0
20. Percent of Strength Retained	
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test CA5 Asphalt Content 4.0

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1861	1846	1836	1864	1867	1866
3. Surface Dry Weight - gm.	1875	1862	1845	1876	1879	1876
4. Immersed Weight - gm.	1066	1050	1048	1068	1068	1064
5. Bulk Volume - c.c.	809.2	806.2	799.2	808.8	810.1	811.4
6. Bulk Specific Gravity - gm/c.c.	2.30	2.29	2.30	2.29	2.29	2.30

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.				2003.5	2026.2	2000.0
8. Immersed Weight - gm.				1094.5	1108.0	1100.1
9. New Bulk Volume - c.c.				909.0	918.2	899.9
10. Volume of Swell - c.c.				99.2	102.1	88.5
11. Weight of Absorbed Water - gm.				127.2	147.1	122.9
12. Percent Swell				11.7	12.5	10.9
13. Percent Water Absorbed				7.1	8.2	6.9

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	880	1010	805	10	13	8
15. Dry Compressive Strength - psi	154	177	141			
16. Soaked Compressive Strength - psi				2	2	1

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.29
18. Dry Strength - psi	157
19. Soaked Strength - psi	2
20. Percent of Strength Retained	1
21. Percent Swell	11.7
22. Percent Water Absorbed	7.4

Data and Results of Immersion-Compression Test

Test CA 7 Asphalt Content 5.6

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1878.4	1872.0	1781.3	1876.8	1868.8	1777.3
3. Surface Dry Weight - gm.	1880.0	1874.1	1783.1	1877.3	1870.0	1780.5
4. Immersed Weight - gm.	1075.5	1074.5	1020.8	1074.6	1069.9	1019.4
5. Bulk Volume - c.c.	804.5	799.6	762.3	802.7	810.1	761.1
6. Bulk Specific Gravity - gm/c.c.	2.34	2.34	2.33	2.34	2.32	2.33

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.				1978.5	1972.6	1889.7
8. Immersed Weight - gm.				1101.9	1083.9	1044.0
9. New Bulk Volume - c.c.				876.6	888.7	845.7
10. Volume of Swell - c.c.				73.9	78.6	84.6
11. Weight of Absorbed Water - gm.				101.2	102.6	109.2
12. Percent Swell				9.2	9.7	11.1
13. Percent Water Absorbed				5.7	5.8	6.5

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	675	780	825	55	43	40
15. Dry Compressive Strength - psi	108	136	144			
16. Soaked Compressive Strength - psi				10	8	7

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.34
18. Dry Strength - psi	129
19. Soaked Strength - psi	8
20. Percent of Strength Retained	6
21. Percent Swell	10.0
22. Percent Water Absorbed	6.0

Data and Results of Immersion-Compression Test

Test CA3(non-aerated) Asphalt Content 2.4

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1820.3	1765.8	1702.8	1880.0	2033.6	1736.8
3. Surface Dry Weight - gm.	1847.3	1797.8	1723.6	1913.9	2051.2	1748.9
4. Immersed Weight - gm.	1027.5	1000.5	963.7	1045.5	1148.1	977.1
5. Bulk Volume - c.c.	819.8	797.3	759.9	868.4	903.1	771.8
6. Bulk Specific Gravity - gm/c.c.	2.22	2.22	2.24	2.26	2.25	2.25

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	0*	0*	350	0*	260	
15. Dry Compressive Strength - psi			61		46	
16. Soaked Compressive Strength - psi	0	0		0		

*- sample disintegrated

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.24
18. Dry Strength - psi	54
19. Soaked Strength - psi	0
20. Percent of Strength Retained	
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Mix CA5(non-aersted) Asphalt Content 4.6

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1896.2	1807.4	1792.9	1858.4	1930.3	1903.8
3. Surface Dry Weight - gm.	1902.1	1813.6	1796.9	1863.4	1936.5	1908.2
4. Immersed Weight - gm.	1076.3	1029.9	1018.9	1056.8	1096.9	1083.8
5. Bulk Volume - c.c.	825.8	783.7	778.0	806.6	839.9	824.5
6. Bulk Specific Gravity - gm/c.c.	2.30	2.30	2.30	2.30	2.30	2.31

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	480	705	680	29	30	26
15. Dry Compressive Strength - psi	84	123	119			
16. Soaked Compressive Strength - psi				5	5	4

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.30
18. Dry Strength - psi	109
19. Soaked Strength - psi	5
20. Percent of Strength Retained	5
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test CA7(non-aerated) Asphalt Content 5.4

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1790.0	1747.4	1684.0	1861.9	1948.6	1983.2
3. Surface Dry Weight - gm.	1795.2	1751.2	1686.2	1865.0	1951.0	1986.0
4. Immersed Weight - gm.	1018.1	992.5	956.0	1060.8	1104.2	1123.9
5. Bulk Volume - c.c.	775.1	758.7	730.2	804.2	846.8	862.1
6. Bulk Specific Gravity - gm/c.c.	2.31	2.31	2.31	2.32	2.31	2.30

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	425	335	370	55	47	50
15. Dry Compressive Strength - psi	75	59	65			
16. Soaked Compressive Strength - psi				10	8	9

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.31
18. Dry strength - psi	66
19. Soaked Strength - psi	9
20. Percent of Strength Retained	13
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test CB3 Asphalt Content 2.4

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1744.1	1741.3	1749.5	1749.0	1753.2	1752.1
3. Surface Dry Weight - gm.	1775.7	1782.0	1772.6	1774.9	1784.0	1777.1
4. Immersed Weight - gm.	990.0	1004.0	985.0	989.6	991.6	990.3
5. Bulk Volume - c.c.	785.7	778.0	787.6	785.3	792.4	786.8
6. Bulk Specific Gravity - gm/c.c.	2.23	2.24	2.22	2.23	2.22	2.23

II AFTER IMMERSION 4 DAYS AT 100°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - kg.	0*	0*	0*	550	745	840
15. Dry Compressive Strength - psi				96	130	147
16. Soaked Compressive Strength - psi	0	0	0			

*- sample disintegrated

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.23
18. Dry Strength - psi	124
19. Soaked Strength - psi	0
20. Percent of Strength Retained	
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test CB5 Asphalt Content 4.0

I AFTER CURING 24 HOURS AT 140°F.	1	2	3	4	5	6
1. Sample Number	1789.2	1813.1	1835.0	1818.2	1832.0	1842.5
2. Dry Weight - gm.	1799.6	1821.1	1844.4	1827.7	1841.1	1850.8
3. Saturated Dry Weight - gm.	1029.0	1042.5	1056.9	1046.0	1053.4	1056.8
4. Immersed Weight - gm.	770.6	778.6	787.5	781.7	787.7	794.0
5. Bulk Volume - c.c.	2.32	2.33	2.33	2.33	2.33	2.33
6. Bulk Specific Gravity - gm/c.c.						

II AFTER IMMERSION 4 DAYS AT 140°F.						
7. Weight in Air - gm.	1938.3	1957.3	1982.0			
8. Immersed Weight - gm.	1082.6	1102.5	1114.9			
9. New Bulk Volume - c.c.	855.7	854.8	867.1			
10. Volume of Swell - c.c.	85.1	76.2	79.6			
11. Weight of Absorbed Water - gm.	138.7	136.2	137.6			
12. Percent Swell	11.0	9.8	10.1			
13. Percent Water Absorbed	8.1	7.8	7.8			

III COMPRESSIVE STRENGTH						
14. Maximum Load - Kg.	18	21	20	890	930	950
15. Dry Compressive Strength - psi				155	162	166
16. Soaked Compressive Strength - psi	3	4	4			

IV AVERAGE VALUES	
17. Bulk Specific Gravity - gm/c.c.	2.33
18. Dry Strength - psi	161
19. Soaked Strength - psi	4
20. Percent of Strength retained	3
21. Percent Swell	10.3
22. Percent Water Absorbed	7.9

Data and Results of Immersion-Compression Test

Test CB7 Asphalt Content 5.6

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1885.6	1914.2	1901.5	1834.6	1879.4	1814.5
3. Surface Dry Weight - gm.	1887.8	1916.5	1904.3	1837.2	1884.0	1817.6
4. Immersed Weight - gm.	1079.8	1095.2	1088.5	1051.1	1076.2	1040.1
5. Bulk Volume - c.c.	808.0	821.3	815.8	786.1	807.8	777.5
6. Bulk Specific Gravity - gm/c.c.	2.34	2.33	2.33	2.33	2.32	2.34

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1976.0	2020.4	1997.0			
8. Immersed Weight - gm.	1094.5	1126.7	1110.1			
9. New Bulk Volume - c.c.	881.5	893.7	886.9			
10. Volume of Swell - c.c.	73.5	72.4	71.1			
11. Weight of Absorbed Water - gm.	88.2	103.9	95.5			
12. Percent Swell	9.1	8.8	8.7			
13. Percent Water Absorbed	5.0	5.7	5.3			

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	41	51	39	490	585	580
15. Dry Compressive Strength - psi				86	102	101
16. Soaked Compressive Strength - psi	7	9	7			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.33
18. Dry Strength - psi	96
19. Soaked Strength - psi	8
20. Percent of Strength Retained	8
21. Percent Swell	8.9
22. Percent Water Absorbed	5.4

Data and Results of Immersion-Compression Test

Test HA3 Asphalt Content 3.1

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1810.9	1790.2	1791.6	1805.6	1816.5	1865.1
3. Surface Dry Weight - gm.	1813.0	1795.7	1808.4	1812.6	1825.8	1873.0
4. Immersed Weight - gm.	1006.0	979.6	954.3	991.2	995.0	1022.0
5. Bulk Volume - c.c.	807.0	816.1	854.1	821.4	830.8	851.0
6. Bulk Specific Gravity - gm/c.c.	2.24	2.19	2.16	2.20	2.19	2.19

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	240	242	137	1970	1400	1750
15. Dry Compressive Strength - psi				345	245	306
16. Soaked Compressive Strength - psi	42	42	24			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.19
18. Dry Strength - psi	299
19. Soaked Strength - psi	35
20. Percent of Strength Retained	12
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test No 5 Asphalt Content 4.8

I AFTER CURING 24 HOURS AT 120°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1816.0	1812.2	1801.4	1843.6	1808.9	1782.8
3. Surface Dry Weight - gm.	1823.2	1820.8	1817.0	1849.8	1819.0	1793.8
4. Immersed Weight - gm.	1013.9	1012.3	1012.4	1027.5	1013.9	981.7
5. Bulk Volume - c.c.	809.3	808.5	804.6	822.3	805.1	812.1
6. Bulk Specific Gravity - gm/c.c.	2.24	2.24	2.24	2.24	2.24	2.20

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	3450	3240	2930	465	280	290
15. Dry Compressive Strength - psi	604	567	512			
16. Soaked Compressive Strength - psi				82	49	52

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.24
18. Dry Strength - psi	561
19. Soaked Strength - psi	61
20. Percent of Strength Retained	9
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Test HA7 Asphalt Content 7.0

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1983.8	1793.0	1784.4	1836.6	1802.6	1867.4
3. Surface Dry Weight - gm.	1988.4	1796.2	1787.6	1840.7	1807.3	1871.9
4. Immersed Weight - gm.	1124.4	1020.6	1021.2	1044.7	1020.3	1059.3
5. Bulk Volume - c.c.	864.0	775.6	766.4	796.0	787.0	812.6
6. Bulk Specific Gravity - gm/c.c.	2.30	2.31	2.33	2.30	2.29	2.30

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.						
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.						
12. Percent Swell						
13. Percent Water Absorbed						

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	460	2795	710	3210	840	3770
15. Dry Compressive Strength - psi		490		561		660
16. Soaked Compressive Strength - psi	81		124		147	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.30
18. Dry Strength - psi	570
19. Soaked Strength - psi	117
20. Percent of Strength Retained	21
21. Percent Swell	
22. Percent Water Absorbed	

Data and Results of Immersion-Compression Test

Mix HB 3 Asphalt Content 3.0

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1815.9	1820.6	1803.2	1821.4	1787.2	1782.6
3. Surface Dry Weight - gm.	1850.0	1856.2	1857.8	1839.2	1803.8	1815.5
4. Immersed Weight - gm.	1029.0	1031.5	1041.5	1014.0	992.3	1008.0
5. Bulk Volume - c.c.	821.0	824.7	816.3	825.3	811.5	805.5
6. Bulk Specific Gravity - gm/c.c.	2.21	2.21	2.21	2.21	2.20	2.21

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1948.9	1953.0	1933.1			
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.	98.9	96.8	75.3			
12. Percent Swell						
13. Percent Water Absorbed	5.6	5.5	4.6			

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	162	129	108	2325	1970	1525
15. Dry Compressive Strength - psi				407	344	267
16. Soaked Compressive Strength - psi	28	23	19			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.21
18. Dry Strength - psi	339
19. Soaked Strength - psi	23
20. Percent of Strength Retained	7
21. Percent Swell	
22. Percent Water Absorbed	5.1

Data and Results of Immersion-Compression Test

Test HB5 Asphalt Content 5.0

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1803.1	1839.3	1735.5	1820.9	1798.0	1793.0
3. Surface Dry Weight - gm.	1829.5	1864.6	1760.6	1848.0	1825.0	1817.3
4. Immersed Weight - gm.	1019.2	1040.5	984.0	1029.9	1019.9	1013.9
5. Bulk Volume - c.c.	810.3	824.1	776.6	818.1	805.1	803.4
6. Bulk Specific Gravity - gm/c.c.	2.23	2.23	2.23	2.23	2.23	2.23

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1915.8	1954.0	1844.4			
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.	86.3	89.4	83.4			
12. Percent Swell						
13. Percent Water Absorbed	5.0	5.1	5.0			

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	229	273	274	1960	2560	2220
15. Dry Compressive Strength - psi				343	448	388
16. Soaked Compressive Strength - psi	40	48	46			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.23
18. Dry Strength - psi	393
19. Soaked Strength - psi	45
20. Percent of Strength Retained	12
21. Percent Swell	
22. Percent Water Absorbed	5.0

Data and Results of Immersion-Compression Test

Test HB7 Asphalt Content 7.0

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1813.8	1820.1	1819.8	1825.0	1797.8	1794.2
3. Surface Dry Weight - gm.	1823.3	1826.1	1826.8	1830.0	1801.4	1799.5
4. Immersed Weight - gm.	1016.7	1017.2	1014.0	1017.8	1007.0	1007.3
5. Bulk Volume - c.c.	806.6	808.9	814.8	812.2	794.4	792.2
6. Bulk Specific Gravity - gm/c.c.	2.25	2.25	2.24	2.25	2.26	2.26

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1896.3	1892.8		1913.2		
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.	73.0	66.7		83.0		
12. Percent Swell						
13. Percent Water Absorbed	4.3	3.9		4.9		

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	570	580	1770	370	1700	1650
15. Dry Compressive Strength - psi			310		297	289
16. Soaked Compressive Strength - psi	99	102		65		

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.25
18. Dry Strength - psi	299
19. Soaked Strength - psi	89
20. Percent of Strength Retained	30
21. Percent Swell	
22. Percent Water Absorbed	4.4

Data and Results of Immersion-Compression Test

Test CFA3 Asphalt Content 3.8

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1783.0	1797.2	1796.0	1827.8	1818.2	1799.0
3. Surface Dry Weight - gm.	1784.2	1798.9	1797.0	1829.6	1819.8	1800.5
4. Immersed Weight - gm.	994.2	1002.4	996.8	1015.2	1005.1	998.0
5. Bulk Volume - c.c.	790.0	796.5	800.2	814.4	814.7	802.5
6. Bulk Specific Gravity - gm/c.c.	2.26	2.26	2.25	2.25	2.23	2.24

II AFTER IMMERSION 4 DAYS AT 100°F.

7. Weight in Air - gm.	1881.0		1901.1		1924.6	
8. Immersed Weight - gm.	1073.1		1080.1		1091.8	
9. New Bulk Volume - c.c.	807.9		821.0		832.8	
10. Volume of Swell - c.c.	17.9		20.8		18.1	
11. Weight of Absorbed Water - gm.	98.0		104.1		104.8	
12. Percent Swell	2.3		2.6		2.2	
13. Percent Water Absorbed	5.8		6.1		6.1	

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	116	2050	100	1870	114	1560
15. Dry Compressive Strength - psi		359		327		273
16. Soaked Compressive Strength - psi	20		18		20	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.25
18. Dry Strength - psi	318
19. Soaked Strength - psi	19
20. Percent of Strength Retained	6
21. Percent Swell	2.4
22. Percent Water Absorbed	6.0

Data and Results of Immersion-Compression Test

Test CFA 5 Asphalt Content 5.2

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1691.9	1740.2	1747.6	1779.7	1772.2	1779.8
3. Surface Dry Weight - gm.	1692.8	1741.9	1748.5	1780.8	1773.8	1781.2
4. Immersed Weight - gm.	927.3	955.7	959.7	975.5	970.7	978.1
5. Bulk Volume - c.c.	765.5	786.2	788.8	805.3	803.1	803.1
6. Bulk Specific Gravity - gm/c.c.	2.21	2.22	2.22	2.21	2.21	2.22

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1780.1	1833.2	1835.9			
8. Immersed Weight - gm.	1004.7	1031.3	1034.2			
9. New Bulk Volume - c.c.	775.4	801.9	801.7			
10. Volume of Swell - c.c.	9.9	15.7	13.4			
11. Weight of Absorbed Water - gm.	87.3	91.3	87.4			
12. Percent Swell	1.5	2.0	1.7			
13. Percent Water Absorbed	5.5	5.6	5.4			

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	110	67	102	1100	1320	1320
15. Dry Compressive Strength - psi				193	232	232
16. Soaked Compressive Strength - psi	19	12	18			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.21
18. Dry Strength - psi	219
19. Soaked Strength - psi	17
20. Percent of Strength Retained	8
21. Percent Swell	1.7
22. Percent Water Absorbed	5.5

Data and Results of Immersion-Compression Test

Test CFA7 Asphalt Content 7.1

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1691.3	1691.9	1696.2	1679.9	1691.8	1702.9
3. Surface Dry Weight - gm.	1692.5	1694.0	1697.8	1681.3	1694.1	1705.4
4. Immersed Weight - gm.	907.0	905.3	915.6	900.2	910.4	923.9
5. Bulk Volume - c.c.	785.5	788.7	782.2	781.1	782.7	781.4
6. Bulk Specific Gravity - gm/c.c.	2.16	2.15	2.17	2.15	2.16	2.18

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.				1769.2	1782.3	1789.2
8. Immersed Weight - gm.				976.6	988.0	996.6
9. New Bulk Volume - c.c.				792.6	794.3	792.6
10. Volume of Swell - c.c.				11.5	11.6	11.2
11. Weight of Absorbed Water - gm.				87.9	89.2	83.8
12. Percent Swell				1.5	1.5	1.4
13. Percent Water Absorbed				5.7	5.7	5.4

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	910	790	890	80	70	80
15. Dry Compressive Strength - psi	159	138	156			
16. Soaked Compressive Strength - psi				14	12	14

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.16
18. Dry Strength - psi	151
19. Soaked Strength - psi	13
20. Percent of Strength Retained	9
21. Percent Swell	1.5
22. Percent Water Absorbed	5.6

Data and Results of Immersion-Compression Test

Test CFB3 Asphalt Content 3.2

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1780.2	1840.7	1785.2	1797.5	1779.1	1774.9
3. Surface Dry Weight - gm.	1782.3	1842.4	1786.0	1799.4	1780.9	1776.8
4. Immersed Weight - gm.	990.1	1020.3	994.1	997.5	981.6	982.9
5. Bulk Volume - c.c.	792.2	822.1	793.9	801.9	799.3	793.9
6. Bulk Specific Gravity - gm/c.c.	2.25	2.24	2.25	2.24	2.23	2.24

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1870.0		1877.1		1872.8	
8. Immersed Weight - gm.	1066.1		1070.3		1062.8	
9. New Bulk Volume - c.c.	803.9		806.8		810.0	
10. Volume of Swell - c.c.	11.7		12.9		10.7	
11. Weight of Absorbed Water - gm.	87.7		89.1		91.9	
12. Percent Swell	1.5		1.6		1.3	
13. Percent Water Absorbed	5.2		5.2		5.4	

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	137	1730	145	1805	138	1700
15. Dry Compressive Strength - psi		302		316		298
16. Soaked Compressive Strength - psi	24		25		24	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.25
18. Dry Strength - psi	305
19. Soaked Strength - psi	24
20. Percent of Strength Retained	8
21. Percent Swell	1.5
22. Percent Water Absorbed	5.3

Data and Results of Immersion-Compression Test

Test CFB5 Asphalt Content 5.1

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1749.1	1741.1	1741.2	1748.0	1776.1	1746.2
3. Surface Dry Weight - gm.	1750.7	1742.6	1743.0	1749.3	1777.3	1748.8
4. Immersed Weight - gm.	947.4	946.3	944.9	948.1	962.9	950.2
5. Bulk Volume - c.c.	803.3	796.3	798.1	801.2	814.4	798.6
6. Bulk Specific Gravity - gm/c.c.	2.18	2.19	2.18	2.18	2.18	2.19

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1839.4	1828.9	1830.8			
8. Immersed Weight - gm.	1028.1	1026.1	1025.8			
9. New Bulk Volume - c.c.	811.3	802.8	805.0			
10. Volume of Swell - c.c.	8.0	6.5	6.9			
11. Weight of Absorbed Water - gm.	88.7	87.3	87.8			
12. Percent Swell	1.0	0.8	0.9			
13. Percent Water Absorbed	5.4	5.4	5.4			

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	100	117	100	1300	1360	1260
15. Dry Compressive Strength - psi				228	238	220
16. Soaked Compressive Strength - psi	18	20	18			

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.18
18. Dry Strength - psi	229
19. Soaked Strength - psi	19
20. Percent of Strength Retained	8
21. Percent Swell	0.9
22. Percent Water Absorbed	5.4

Data and Results of Immersion-Compression Test

Test CFB7 Asphalt Content 7.2

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1700.5	1720.0	1706.9	1724.7	1702.6	1696.7
3. Surface Dry Weight - gm.	1702.2	1721.8	1708.1	1726.1	1703.9	1698.2
4. Immersed Weight - gm.	912.3	921.3	919.1	927.5	914.6	909.8
5. Bulk Volume - c.c.	789.9	800.5	789.0	798.6	789.3	788.4
6. Bulk Specific Gravity - gm/c.c.	2.16	2.15	2.16	2.16	2.16	2.15

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.				1812.2	1789.2	1783.9
8. Immersed Weight - gm.				1007.5	994.1	989.1
9. New Bulk Volume - c.c.				804.7	795.1	794.8
10. Volume of Swell - c.c.				6.1	5.8	6.4
11. Weight of Absorbed Water - gm.				86.1	85.3	85.7
12. Percent Swell				0.8	0.7	0.8
13. Percent Water Absorbed				5.4	5.4	5.5

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	1110	1050	960	90	73	78
15. Dry Compressive Strength - psi	194	184	168			
16. Soaked Compressive Strength - psi				16	13	14

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.16
18. Dry Strength - psi	175
19. Soaked Strength - psi	14
20. Percent of Strength Retained	8
21. Percent Swell	0.8
22. Percent Water Absorbed	5.4

Data and Results of Immersion-Compression Test

Mat HFA3 Asphalt Content 3.3 %

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1749.9	1786.4	1789.4	1765.8	1775.9	1786.3
3. Surface Dry Weight - gm.	1762.8	1799.6	1808.8	1777.4	1789.8	1799.7
4. Immersed Weight - gm.	980.2	998.5	1004.9	979.3	980.2	995.1
5. Bulk Volume - c.c.	782.6	801.1	803.9	798.1	809.6	804.6
6. Bulk Specific Gravity - gm/c.c.	2.23	2.23	2.23	2.21	2.20	2.23

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.		1943.9	1937.8		1926.4	
8. Immersed Weight - gm.						
9. New Bulk Volume - c.c.						
10. Volume of Swell - c.c.						
11. Weight of Absorbed Water - gm.		144.3	129.0		136.6	
12. Percent Swell						
13. Percent Water Absorbed		8.4	7.5		8.0	

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	2735		70	1880	105	2290
15. Dry Compressive Strength - psi	479			328		401
16. Soaked Compressive Strength - psi			12		18	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.22
18. Dry Strength - psi	403
19. Soaked Strength - psi	15
20. Percent of Strength Retained	4
21. Percent Swell	
22. Percent Water Absorbed	7.9

Data and Results of Immersion-Compression Test

Test HFA5 Asphalt Content 5.3 %

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	18168	18041	18930	18390	18619	18572
3. Surface Dry Weight - gm.	18219	18128	19032	18493	18719	18665
4. Immersed Weight - gm.	10332	10209	10759	10419	10519	10498
5. Bulk Volume - c.c.	788.6	791.9	827.3	807.4	820.0	817.7
6. Bulk Specific Gravity - gm/c.c.	2.31	2.28	2.29	2.28	2.27	2.29

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	19251	19163			19730	
8. Immersed Weight - gm.	10649	10645			11076	
9. New Bulk Volume - c.c.	860.2	851.8			865.4	
10. Volume of Swell - c.c.	71.6	59.9			45.4	
11. Weight of Absorbed Water - gm.	103.2	103.5			101.1	
12. Percent Swell	9.1	7.6			5.5	
13. Percent Water Absorbed	6.0	6.1			5.7	

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	233	218	3540	3510	373	4070
15. Dry Compressive Strength - psi			619	616		712
16. Soaked Compressive Strength - psi	41	38			65	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.29
18. Dry Strength - psi	649
19. Soaked Strength - psi	48
20. Percent of Strength Retained	7
21. Percent Swell	7.4
22. Percent Water Absorbed	5.9

Data and Results of Immersion-Compression Test

Test HFA7 Asphalt Content 7.5 %

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1963.3	1823.0	1862.8	1843.0	1841.0	1810.4
3. Surface Dry Weight - gm.	1968.3	1826.8	1867.0	1847.1	1847.0	1813.8
4. Immersed Weight - gm.	1126.7	1046.8	1068.8	1057.3	1056.3	1038.7
5. Bulk Volume - c.c.	841.6	780.0	798.2	789.8	790.7	775.1
6. Bulk Specific Gravity - gm/c.c.	2.33	2.34	2.33	2.34	2.33	2.34

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.				1915.2	1908.1	1876.8
8. Immersed Weight - gm.				1085.5	1088.2	1067.9
9. New Bulk Volume - c.c.				829.7	819.9	808.9
10. Volume of Swell - c.c.				39.9	29.2	33.8
11. Weight of Absorbed Water - gm.				68.1	61.1	63.0
12. Percent Swell				5.1	3.7	4.4
13. Percent Water Absorbed				4.0	3.6	3.7

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	2620	4000	3400	590	900	980
15. Dry Compressive Strength - psi	458	700	612			
16. Soaked Compressive Strength - psi				103	157	171

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.33
18. Dry Strength - psi	590
19. Soaked Strength - psi	144
20. Percent of Strength Retained	24
21. Percent Swell	4.4
22. Percent Water Absorbed	3.8

Data and Results of Immersion-Compression Test

Test HFB3 Asphalt Content 3.1

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1684.6	1711.3	1726.7	1712.8	1726.7	1886.7
3. Surface Dry Weight - gm.	1703.0	1735.2	1750.6	1738.0	1742.3	1907.8
4. Immersed Weight - gm.	938.5	962.8	973.8	958.5	961.8	1051.8
5. Bulk Volume - c.c.	764.5	772.4	776.8	779.5	780.5	856.0
6. Bulk Specific Gravity - gm/c.c.	2.21	2.22	2.22	2.20	2.21	2.20

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.			1825.3*		1868.7	2039.6
8. Immersed Weight - gm.			1018.3		1036.8	1130.1
9. New Bulk Volume - c.c.			807.0		831.9	909.5
10. Volume of Swell - c.c.			30.2		51.4	53.5
11. Weight of Absorbed Water - gm.			74.7		126.4	131.8
12. Percent Swell			3.9		6.6	6.2
13. Percent Water Absorbed			4.5		7.6	7.2

* sample partly disintegrated

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	1720	1220	156	1250	207	173
15. Dry Compressive Strength - psi	305	214		218		
16. Soaked Compressive Strength - psi			27		36	30

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.21
18. Dry Strength - psi	244
19. Soaked Strength - psi	31
20. Percent of Strength Retained	13
21. Percent Swell	6.4
22. Percent Water Absorbed	7.4

[illegible] and Results of Immersion-Compression Test

Test HFB5 Asphalt Content 5.1

I AFTER [illegible] 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1857.3	1864.1	1836.4	1827.2	1834.4	1807.2
3. Surface Dry Weight - gm.	1884.8	1890.5	1863.1	1851.4	1865.0	1831.6
4. Immersed Weight - gm.	1056.9	1060.7	1049.4	1039.5	1046.9	1026.2
5. Bulk Volume - c.c.	827.9	829.8	813.7	811.9	818.1	805.4
6. Bulk Specific Gravity - gm/c.c.	2.24	2.25	2.26	2.25	2.24	2.24

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.	1994.3		1965.0		1961.9	
8. Immersed Weight - gm.	1097.6		1088.1		1085.6	
9. New Bulk Volume - c.c.	896.7		876.9		876.3	
10. Volume of Swell - c.c.	68.8		63.2		58.2	
11. Weight of Absorbed Water - gm.	99.5		101.9		96.9	
12. Percent Swell	8.3		7.9		7.1	
13. Percent Water Absorbed	5.6		5.8		5.6	

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	238	2210	243	2050	239	2690
15. Dry Compressive Strength - psi		388		358		470
16. Soaked Compressive Strength - psi	42		43		42	

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.25
18. Dry Strength - psi	404
19. Soaked Strength - psi	42
20. Percent of Strength Retained	10
21. Percent Swell	7.8
22. Percent Water Absorbed	5.7

Data and Results of Immersion-Compression Test

Test HFB7 Asphalt Content 7.1

I AFTER CURING 24 HOURS AT 140°F.

	1	2	3	4	5	6
1. Sample Number						
2. Dry Weight - gm.	1885	1891	1893	1814	1711	1957
3. Surface Dry Weight - gm.	1890	1897	1899	1820	1717	1965
4. Immersed Weight - gm.	1061	1068	1072	1027	970.8	1106
5. Bulk Volume - c.c.	821.8	828.2	826.9	793.3	746.6	859.5
6. Bulk Specific Gravity - gm/c.c.	2.28	2.28	2.29	2.29	2.30	2.28

II AFTER IMMERSION 4 DAYS AT 120°F.

7. Weight in Air - gm.			1987	2190		2063
8. Immersed Weight - gm.			1106	1057		1142
9. New Bulk Volume - c.c.			880.3	852.9		920.8
10. Volume of Swell - c.c.			53.4	59.6		61.3
11. Weight of Absorbed Water - gm.			88.1	89.3		97.2
12. Percent Swell			6.5	7.5		7.2
13. Percent Water Absorbed			5.0	5.3		5.3

III COMPRESSIVE STRENGTH

14. Maximum Load - Kg.	2585	2460	326	267	2200	196
15. Dry Compressive Strength - psi	452	430			385	
16. Soaked Compressive Strength - psi			57	47		34

IV AVERAGE VALUES

17. Bulk Specific Gravity - gm/c.c.	2.29
18. Dry Strength - psi	422
19. Soaked Strength - psi	46
20. Percent of Strength Retained	11
21. Percent Swell	7.0
22. Percent Water Absorbed	5.2